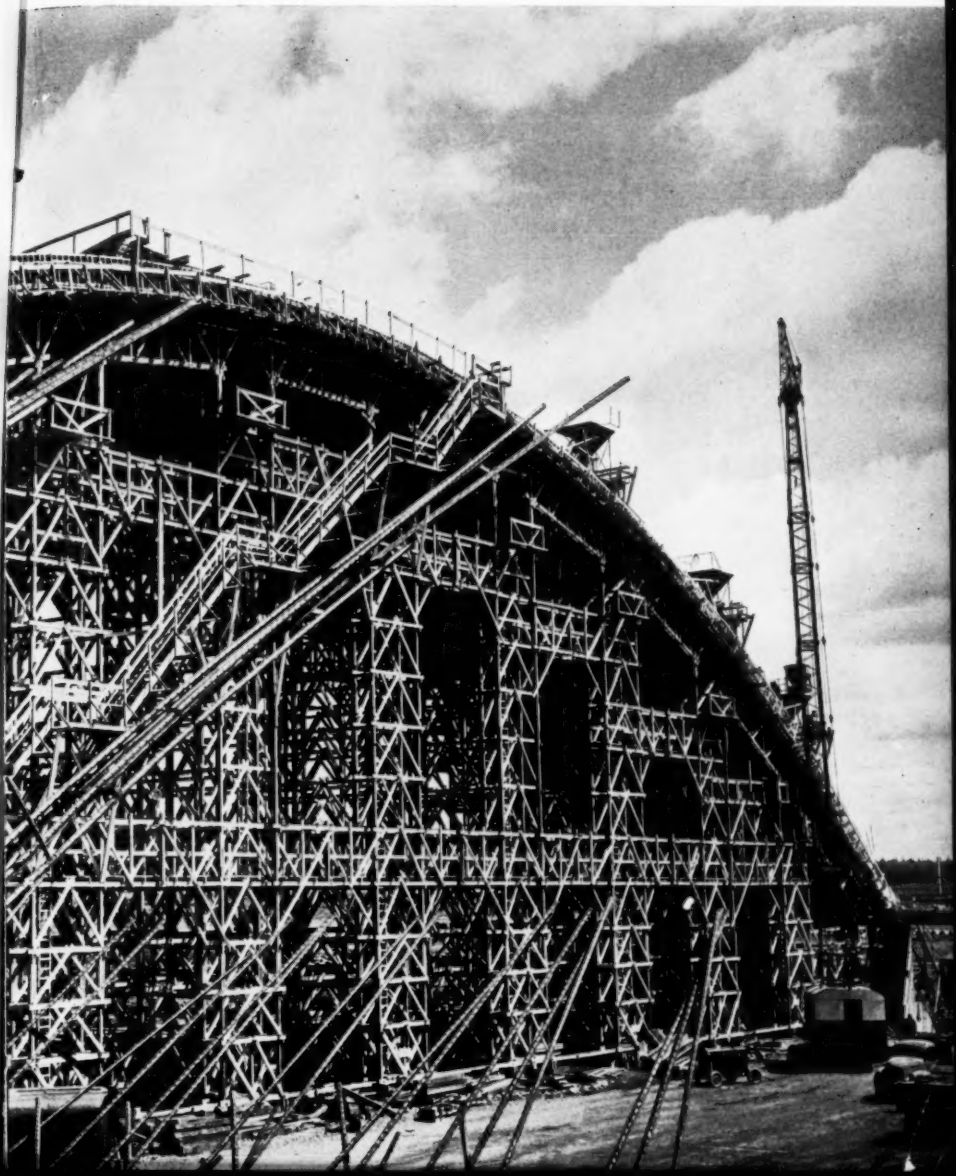
ACI News Letter

Vol. 21 No. 6 JOURNAL of the AMERICAN CONCRETE INSTITUTE February 1950



FROM THE BOTTOM UP...

...applies both to accepted construction procedures and to the method used in making Prepakt concrete. By grouting compacted coarse aggregate from the bottom, Prepakt eliminates the possibility of air pockets and honeycomb in the mass, and provides the finished concrete with high density and resistance to all deterioration factors.

From start to finish the Prepakt engineering and field production organization is geared for efficient production and placement of structural concrete. Prepakt representatives will gladly demonstrate the many advantages of Prepakt concrete upon your request to the office nearest you.

Intrusion-Prepakt wishes to announce the opening of a new regional office in the Seattle Savings and Trust Company Building, Seattle, Washington.

PREPAKT

THE CONCRETE WITH EXTREME DURABILITY

INTRUSION-PREPAKT, INC. THE PREPAKT CONCRETE CO.

CHICAGO · TORONTO **CLEVELAND 14, OHIO** SEATTLE · PHILADELPHIA

ACI

News Letter Contents

February 1950

On the Cover—The falsework superstructure used in constructing the long-span concrete arch hangar at Limestone Air Force Base. The construction of the hangar is described in the paper by John E. Allen on p. 405.

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Time Table for 46th Annual Convention

February 20-22, 1950

Edgewater Beach Hotel

Chicago

Monday, February 20

9 a.m.—Registration begins

ACI committee meetings in the morning are open to visitors only by consent of committee chairman.

Members and others not engaged in technical committee meetings have the opportunity to meet and to discuss mutual interests with old friends and acquaintances and to make new ones—not the least of the benefits of ACI conventions.

2 p.m.—First general convention session—Ballroom

Tuesday, February 21

(Concurrent sessions will be held morning and afternoon in the Ballroom and East Lounge.)

9 a.m.—Convention session—Ballroom

9 a.m.—Convention session—East Lounge

12:30 p.m.—General Luncheon—Marine Dining Room (by tickets previously purchased)

2:30 p.m.—Convention session—Ballroom

2:30 p.m.—Convention session—East Lounge

Wednesday, February 22

9 a.m.—Open meeting of ACI Committee 115, Research—Ballroom

2 p.m.—Final convention session—Ballroom

(See following pages for program)

ACI Day by Day in Chicago

February 20-22, 1950

Edgewater Beach Hotel

CONVENTION REGISTRATION

Registration fee—$1.00

(All **registered** nonmember visitors are welcome at general sessions)

Ballroom Foyer

9 a.m.—5 p.m., DAILY

MONDAY, TUESDAY and WEDNESDAY

FEBRUARY 20, 21, 22, 1950

in charge of

ETHEL B. WILSON, Assistant Treasurer

TECHNICAL COMMITTEE MEETINGS

See bulletin board at registration desk for time and location.

FIRST CONVENTION SESSION

2 p.m., MONDAY, FEBRUARY 20

Ballroom

President HERBERT J. GILKEY, Presiding

Opening remarks

Appointment of tellers to canvass annual election ballots

INSPECTION

J. W. KELLY, Session Leader

"The Inspector"

MILES N. CLAIR

Thompson and Lichtner Co., Inc. Boston, Mass.

The qualifications and functions of the inspector will be discussed. (Tentatively scheduled for May Journal)

"Inspection and Testing of Materials"

NICOLAAS T. F. STADTFELD

Board of Water Supply, New York City

See ACI Journal, December 1949, wherein Mr. Stadtfeld describes the care taken in the inspection of 7 million barrels of cement and shows above all the necessity for manufacturing control of the cement clinker.

"Inspection of Building Construction"

LEONARD E. DUNLAP

Carr and Wright, Inc., Chicago

The author will describe, from the viewpoint of an architect, the number of factors involved in the inspection of architectural and structural concrete. (Tentatively scheduled for April Journal)

"Inspection and Control of Concrete for Highway and Bridge Construction"

H. W. RUSSELL

Illinois Division of Highways, Springfield, Ill.

See ACI Journal, February 1950, in which the author describes the inspection procedures and personnel assignments used by the Illinois Division of Highways.

"Inspection of Mass and Related Concrete Construction"

LEWIS H. TUTHILL

U. S. Bureau of Reclamation, Denver, Colo.

See ACI Journal, January 1950. This paper points out that inspection of any kind can be no more effective than that permitted by the specifications and, particularly, by the established job standard of inspection. No distinction is made between the usual inspection activities and those associated with concrete control since both have the same objective.

"The Contractor's Viewpoint on Inspection"

NOMER GRAY

Walter Kidde Contractors, Inc., New York City

and DONALD C. ANDREWS

Turner Construction Co., New York City

The interests of owner and contractor will be compared and recommendations made on specifications, personal qualifications of the inspector, and methods of minimizing friction with the contractor. (Tentatively scheduled for the Mar. Journal)

"Inspection of Ready-Mixed Concrete"

E. L. HOWARD

Pacific Coast Aggregates, Inc., San Francisco, Calif.

The author will list inspection problems involving the control of aggregate grading and rapid changes in mix design in the production of ready-mixed concrete to meet many different specifications. (Tentatively scheduled for June Journal)

"Summary of Inspection Practice"

R. B. YOUNG and WILFRID SCHNARR

Hydro-Electric Power Commission of Ontario, Toronto, Ont., Canada

The authors will summarize the salient points brought out during the session. (Tentatively scheduled for June Journal)

TUESDAY, FEBRUARY 21

Concurrent sessions will be held morning and afternoon in the **Ballroom** and **East Lounge.**

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9 a.m. TUESDAY—Ballroom

Vice-President HARRY F. THOMSON, Chairman

ACI BUILDING CODE STUDIES

CHESTER L. POST, Session Leader

"Deformed Reinforcing Bars"

RAYMOND C. REESE

Consulting Engineer, Toledo, Ohio

The author will trace the development of reinforcing bars from the original plain round or square forms to the present improved deformed patterns and emphasize the need for a revision of the ACI Code. (Tentatively scheduled for May Journal)

"Two-Way Slabs—Suggested Changes in the Code"

C. P. SIESS and N. M. NEWMARK

University of Illinois, Urbana, Ill.

A new design specification for two-way concrete floor slabs will be suggested by the authors. (Tentatively scheduled for June Journal)

PROBLEMS COMMON TO COMMITTEES 208, Bond Stress; 318, Standard Building Code; 323, Prestressed Reinforced Concrete; 324, Precast Reinforced Concrete Structures and 711, Precast Floor Systems for Houses

F. N. MENEFEE, Session Leader

"Spacing of Moment Bars in Precast Joists"

F. N. MENEFEE

University of Michigan, Ann Arbor, Mich.

and H. L. KINNEAR

University of Virginia, Charlottesville, Va.

Tests of precast joists whose bar spacing is less than that provided for in the ACI Code will be described. Theoretical considerations and results of actual tests will be given for joists with ¾-in. spacing between moment bars. (Tentatively scheduled for Apr. Journal)

C. D. WILLIAMS and F. BROMILOW, University of Florida, will present a prepared discussion on the subject, based on bond tests made on precast joists.

"Proposed Specifications for Minimum Bar Spacing and Protective Cover in Precast Concrete Framing Members"

A. AMIRIKIAN

Navy Dept., Bureau of Yards and Docks, Washington, D. C.

The author will recommend a new basis for specifying bar spacing and cover in the form of proposed specifications applicable to precast concrete work, and will discuss some of the considerations justifying the suggested specifications. (Tentatively scheduled for Apr. Journal)

"Extent and Acceptability of Cracking in Precast Concrete Framing Members"

A. AMIRIKIAN

One of the important questions arising in concrete prefabrication concerns the acceptability of cracked framing members. The author will propose an acceptability clause which would give practical definitions of cracks and specify limitations for acceptability. (Tentatively scheduled for May Journal)

"The Patent Status of Prestressed Concrete and the Relationship of Prestressed Concrete to Building Codes"

CURZON DOBELL

Preload Corp., New York City

The author will give a brief account of the more important groups of prestressed concrete patents and their uses and will present a synopsis of codes in preparation in various countries. He will cite the problems to be overcome in drafting a prestressed code for domestic use. (Tentatively scheduled for Apr. Journal)

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9 a.m. TUESDAY—East Lounge

Vice-President F. H. JACKSON, Chairman

STRUCTURAL DESIGN OF CONCRETE PAVEMENTS

L. W. TELLER, Session Leader

"Influence of Subgrades and Bases on Design of Rigid Pavements"

KENNETH B. WOODS

Purdue University, Lafayette, Ind.

See ACI Journal, January 1950. This paper includes a lengthy bibliography on the subject of subgrades and bases. Indications are that the structural capacity of rigid pavements can be improved by the use of location procedures which utilize the best in topographic position and subgrade soil textures.

"The Problem of Slab Dimensions"

A. T. GOLDBECK

National Crushed Stone Assn., Washington, D. C.

The dimensions of concrete pavement slabs depend on and are influenced by many factors. The author will tabulate and discuss the factors. (Tentatively scheduled for Apr. Journal)

"Design and Construction of Joints in Concrete Pavements"

WILLIAM VAN BREEMEN

New Jersey State Highway Dept., Trenton, N. J.

and E. A. FINNEY

Michigan State Highway Dept., Lansing, Mich.

The authors will discuss the fundamental conditions involved in the design and construction of joints. Construction recommendations will be given. (Tentatively scheduled for May Journal)

"What Can Steel in Concrete Pavements Do?"

BENGT F. FRIBERG

Granite City Steel Co., Granite City, Ill.

The early progress in the use of reinforcement in pavements will be summarized and recent developments described. (Tentatively scheduled for June Journal)

GENERAL LUNCHEON

12:30 p.m. TUESDAY—Marine Dining Room—For ACI Members and friends who buy their tickets at registration ($2.75 including service)

PRESIDENT GILKEY will make formal presentation of the tokens of awards made by the Board of Direction as follows:

ALFRED E. LINDAU Award for outstanding contributions to reinforced concrete design practice to

FRANK E. RICHART

"for his years of labor toward the improvement of reinforced concrete design practice"

LEONARD C. WASON Medal for the "Most Meritorious Paper" of ACI Proceedings Vol. 45 (December 1948 Journal) to

C. P. SIESS and N. M. NEWMARK

for their paper

"Rational Analysis and Design of Two-Way Concrete Slabs"

LEONARD C. WASON Medal for "Noteworthy Research" to

T. M. KELLY, L. SCHUMAN and F. B. HORNIBROOK

for the work reported in their paper (Proceedings Vol. 45, September 1948 Journal)

"A Study of Alkali-Aggregate Reactivity by Means of Mortar Bar Expansions"

American Concrete Institute CONSTRUCTION PRACTICE AWARD for a "paper of outstanding merit" on concrete construction practice to

S. O. ASPLUND

for his paper in the ACI Journal (January 1949) entitled

"Strengthening Bridge Slabs with Grouted Reinforcement"

Report of Annual Election and Introduction of New Officers

Retiring President's Address by HERBERT J. GILKEY, Iowa State College, Ames, Iowa

2:30 p.m. TUESDAY—Ballroom

President H. J. GILKEY, Chairman

PRESTRESSED, PRECAST and REINFORCED CONCRETE

"Corrosion Protection of Thin Precast Concrete Sections"

D. H. PLETTA, E. F. MASSIE and H. S. ROBINS

Virginia Polytechnic Institute, Blacksburg, Va.

A new electrical resistance technique for measuring the rate of corrosion of steel reinforcing will be described. (Tentatively scheduled for Mar. Journal)

"Prestressed Concrete Construction Procedures"

THOR GERMUNDSSON

Portland Cement Assn., Chicago

The basic concepts of prestressed concrete will be reviewed as well as examples of the application of several systems. (Tentatively scheduled for June Journal)

"A Simple Mathematical Tool for the Solution of Difficult Design Problems"

ALFRED PARME

Portland Cement Assn., Chicago

This paper will deal with the application of finite differences to the solution of those structural problems in which the physical relationships are expressed as a differential equation. (Tentatively scheduled for May Journal)

"Precast Concrete Construction in Britain"

P. G. BOWIE and A. R. COLLINS

Cement and Concrete Assn., London, England

Various precast frames and systems used in construction in Great Britain will be described. (Tentatively scheduled for Mar. Journal)

(In the authors' absence, the paper will be presented by Everett E. Ebling)

"Precast Reinforced Concrete Structures"

C. D. WAILES, JR.

C. D. Wailes Corp., Los Angeles, Calif.

The author will focus attention on the practical application of precast construction in the United States. (Tentatively scheduled for Apr. Journal)

2:30 p.m. TUESDAY—East Lounge

Past President R. F. BLANKS, Chairman

ADMIXTURES

W. T. MORAN, Session Leader

"Use of Admixtures to Increase Resistance to Freezing and Thawing"

F. H. JACKSON

Bureau of Public Roads, Washington, D. C.

The relative merits of admixtures and interground agents will be considered and optimum ranges of air content for different structural uses stated. (Tentatively scheduled for May Journal)

"Use of Admixtures in Concrete Products"

BRUCE E. FOSTER

National Bureau of Standards, Washington, D. C.

The author will discuss the effects and precautions in the use of additives in connection with concrete products. (Tentatively scheduled for May Journal)

"Use of Admixtures in Correcting Grading Deficiencies"

T. C. POWERS

Portland Cement Assn., Chicago

The factors affecting bleeding characteristics and workability of fresh concrete will be reviewed. (Tentatively scheduled for May Journal)

"Use of Admixtures in Counteracting Alkali-Aggregate Reactions"

W. T. MORAN

The author will review the present knowledge of this subject. (Tentatively scheduled for May Journal)

"Use of Admixtures as Integral Dampproofing Materials"

BRUCE E. FOSTER

The mechanics of the passage of water through concrete and the effect of dampproofing materials on permeability will be discussed. (Tentatively scheduled for May Journal)

GENERAL TOPICS

"Cost of Long-Span Reinforced Concrete Arch Roofs"

CHARLES S. WHITNEY

Ammann and Whitney, New York City

The author will describe five long-span shell construction jobs and compare the costs of concrete structures with those of timber and steel. (Tentatively scheduled for June Journal)

"Effect of High Velocity Water on Bonneville Dam Concrete"

R. R. CLARK

Office of the District Engineer, Dept. of the Army, Portland, Ore.

The author will compare the progressive erosion which has taken place in the stilling basin with that anticipated in the design, together with the repairs that have been made and the effect on the repairs by the action of high-velocity water. (Tentatively scheduled for May Journal)

(In the author's absence, the paper will be presented by W. H. Price.)

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9 a.m. WEDNESDAY, FEBRUARY 22

Ballroom

The Annual Open Session of ACI Committee 115, Research

STEPHEN J. CHAMBERLIN, Chairman

GEORGE W. WASHA, Secretary

This convention feature, inaugurated in 1937, has been increasingly popular. It differs from general sessions in that the information is not released for publication. Request is made that proceedings be regarded as CONFIDENTIAL. The projects to be reported upon are not far enough along for publication; none of them is past the active stage and some are in only the preliminary stage. Some may be published in the near future; some perhaps not for years. Attendance at the session is the only sure way of being in on these previews of the many future developments in concrete. (A stenotype report will be made for use at Institute headquarters and in committee records only. This report is NOT for release.)

Plastic flow of reinforced concrete slabs and beams

Measurement of consistency of concrete by ball penetration

A new technique for determining stresses in steel reinforcement as a result of volume changes

Instruments for measuring internal strain in concrete

Measurement of strain within a concrete mass

Studies of concrete subjected to changes in temperature

Influence of thermal properties of aggregates on durability of concrete

Field experiences with the soniscope inspecting concrete

Diagonal tension in reinforced concrete beams

Strength of concrete under triaxial loading

Tests of eccentrically loaded columns

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2 p.m. WEDNESDAY, FEBRUARY 22

Ballroom

Panel Discussion Based on Questions submitted by ACI Members

C. H. SCHOLER, Chairman

A. AMIRIKIAN
MILES N. CLAIR
RAYMOND E. DAVIS
BRYANT MATHER
DOUGLAS McHENRY
W. T. MORAN
N. M. NEWMARK
T. C. POWERS
W. H. PRICE
A. G. TIMMS

Panel discussion of questions and problems about concrete, concrete construction and related materials. The session promises to be of top interest with every reason to believe that differences of opinion will make it lively and worthwhile.

CHICAGO CONVENTION COMMITTEE

(In charge of local arrangements)

HARRY DELZELL, Chairman

FRANK W. EDWARDS, H. F. GONNERMAN, WILLIAM T. F. HOOPER, JR., JOHN F. SEIFRIED, C. P. SIESS, H. F. THOMSON, W. W. WALLACE

ACI inaugurates microfilm service for ACI members and subscribers

ACI has recently entered into an arrangement with University Microfilms, Ann Arbor, Mich., whereby issues of the ACI JOURNAL can be made available in microfilm to Members and subscribers—not to the general public.

One of the most pressing problems facing public and private libraries today is that of providing adequate storage space.

The ACI JOURNAL falls in that class of materials—periodicals—which gives particular difficulty, for the number of JOURNALS is great and the bulk large. According to "The Problem of Periodical Storage in Libraries," published by University Microfilms, journals in science and technology ordinarily have the greatest use during the first two or three years following publication. Much of the material in the ACI JOURNAL is used for reference far beyond that time limit, but the library is faced with the physical limitation of providing storage space.

Quoting the bulletin mentioned: "Studies have shown that in the average library the estimated storage cost of periodical material is approximately 20 cents per volume per year. Therefore, a library shelving 5000 titles a year incurs an annual storage charge that increases by $1000 each year. This charge cumulated over a 10-year period is $55,000 and over a 30-year period totals $465,000 for storage costs of periodicals acquired during the same period!"

Without considering the question of how many titles a library should be expected to preserve, it is apparent that if many periodicals such as the ACI JOURNAL are to be kept available for future use, it must be without such excessive storage costs.

The Institute finds a possible solution of this problem through microfilm. If multiple copies are made, the reduction ratio controlled, and careful attention given all production costs and techniques, it is possible to provide microfilm copies at a cost about equal to a standard library binding of the same volume.

The newly inaugurated service would operate as follows in the average library: (1) the regular issue of the ACI JOURNAL would be received monthly on the regular subscription; (2) another subscription for microfilm would be placed with University Microfilms for a microfilm copy to be delivered at the end of the volume year (after publication of the December, Part 2, JOURNAL carrying discussion of papers published in previous JOURNAL issues September through June); and (3) the original paper copies of the JOURNAL are used unbound, until after the period of greatest use or the copies worn out—depending on the amount of use and storage space available. The paper copies then would be discarded and the microfilm copy substituted.

Since the cost of the film is roughly equal to the binding cost which would be spent on the paper copies ($3 to $5 if enough Members and subscribers order microfilm), no additional expense is incurred in acquiring the film. Use of microfilm in this way can save 94 percent of the storage costs of the paper edition.

The microfilm will be available to individuals as well as libraries at any time after the completion of a volume year, and does not have to be ordered at the same time the JOURNAL subscription is placed with the Institute.

The usual bound volume of the ACI JOURNAL **will still be available** as before, through Institute headquarters.

Further information on either bound volumes or microfilm copies can be obtained from Institute headquarters or from University Microfilms, Ann Arbor, Mich.

CONCRETE BY PIPE LINE

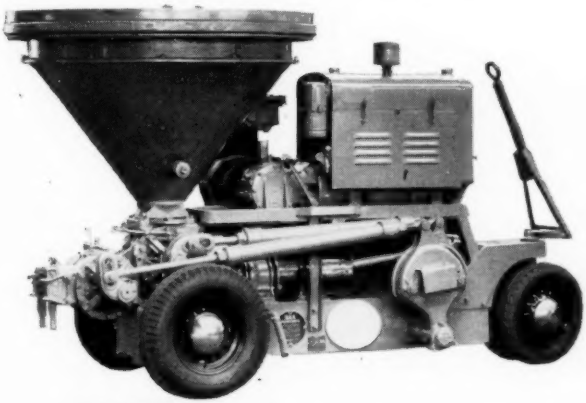
The Low-Cost Method of Placing Quality Concrete!

Rex Pumpcrete offers the lowest cost method of placing concrete of improved quality. The "concrete by pipe line method," properly employed, will assure the best job structurally at far lower cost as compared to other placement methods.

Segregation of the mix is completely eliminated by the conditioning hopper and direct pumping into the forms. Discharge into the forms is more readily controlled. Proof of Pumpcrete's quality placement is evidenced by the fact that, where quality is paramount, leading engineering organizations specify Pumpcrete.

In addition to its quality placement, Rex Pumpcrete permits greater flexibility and convenience in job set-ups. Concrete is moved "from here to there" in one operation, thus eliminating much of the preparatory work, many units of normal high cost equipment and confusion of ordinary placement methods.

It may be that your next job should be a Pumpcrete job. Why not consult your Rex Distributor and get the full story of what Pumpcrete . . . *the pump that pumps concrete through a pipe line* . . . can do for you. Or if you prefer, write directly to Chain Belt Company, 1713 West Bruce Street, Milwaukee 4, Wisconsin.



CHAIN BELT COMPANY REX CONSTRUCTION MACHINERY

Drawing for Reproduction

By C. H. KEARNS, JR. and A. J. PHILBY*

Illustrations are an important part of any manuscript prepared for publication in technical journals. ACI authors will find the following article an excellent guide in the preparation of drawings for publication. It is reprinted with permission from the *Ohio State University Engineering Experiment Station News*, Dec. 1949, p. 16.

During the course of a career in engineering, in research, or in any field which requires the presentation of statistical information in other than tabular form, occasions will arise which demand the preparation of graphic materials for book or journal (technical or trade) publication. An understanding of at least the basic factors involved in the preparation of such material is necessary for an effective presentation of the story. An illustration, properly drawn and reproduced, will attract the reader's eye, increase the interest, and supplement and clarify the text material. A poorly designed and carelessly drawn illustration will confuse the reader, and prompt him to turn to a better prepared and more easily understood article.

Process

The distinct difference in the two words, *duplicate* and *reproduce*, is often overlooked by those engaged in preparing graphic materials. A pencil or ink drawing (on tracing paper or cloth), easily and clearly readable, will produce a satisfactory duplicate. Copies made by blueprint, "ozalid," or similar process are the same size as the original and will substitute readily for it.

Copies of an original drawing can also be made by a photographic or a photostat process. If the copies are the same size as the original, then the process is that of duplication. However, if the copies are smaller than the original, then they are designated as reduced reproductions and must observe certain control factors.

*Instructor and Associate Professor of Engineering Drawing, respectively, Ohio State University, Columbus, Ohio.

Materials

The choice of proper materials for preparing the drawing is of primary importance. Any amount of care in making the zinc cut, the half-tone,† or the multilith plate will not produce a satisfactory reproduction if the original is not in suitable form. Black drawing ink and good quality white drawing paper, with a dull smooth surface and good erasing properties, give the best results. Tracing paper and tracing cloth are also satisfactory media. Lines should be intense enough to contrast sharply with the background. In general, colored inks do not reproduce well, a factor that eliminates the direct use of most prepared graph paper. Pencil drawings, photostats, blueprints, and "ozalid" prints are not suitable for use in the reproduction process.

The original

Drawings for reproduction are usually made larger than they will appear in printed form. This custom introduces a second very important consideration. In drawing the original, care must be taken to avoid use of a line so fine or lettering so small that it will be lost in reduction. The minimum line weight that can be held on a zinc etching or multilith plate is approximately 1/500 of an inch. The smallest readable letter height is 1/20 of an inch. If a chart to be used in a 3-in. column is drawn double size or 6 in. wide, the original should not have lines less than 1/250 of an inch wide or lettering less than 1/10 of an inch high. Line widths may be judged by comparison with

†Editor's note—"line cuts" sometimes are copper and half-tones may be zinc.

Improved Bond Strength for Firm Anchorage in Concrete

Wide, corrosion-inviting cracks in reinforced concrete—all too frequent in the past with reinforcing bars having low bond strength—are unlikely when the improved Bethlehem Reinforcing Bar is used.

The Bethlehem Reinforcing Bar virtually eliminates wide, unsightly cracks in concrete because of its exceptional bond strength. The bar has high, closely-spaced ribs which provide a continuous firm key between steel and concrete. This means low slips at working loads, and the lower the slip of steel in concrete the finer and more closely-spaced will be the resulting tensile cracks.

The Bethlehem Reinforcing Bar is made from new-billet steel in all sizes from $\frac{3}{8}$ in. to $1\frac{1}{4}$ in. It meets all the requirements of ASTM Specification A305-49. Specify bars to meet A305-49 in your next concreting job, to be sure of better control of tensile cracks, as well as improved anchorage between concrete and steel.

BETHLEHEM STEEL

BETHLEHEM STEEL COMPANY, BETHLEHEM, PA.

On the Pacific Coast Bethlehem products are sold by Bethlehem Pacific Coast Steel Corporation

Export Distributor: Bethlehem Steel Export Corporation

Bethlehem Reinforcing Bars

"Mann's Line Gage"* and letter heights established with a scale or with one of the available guide line instruments. Fig. 1 illustrates the effect of size reduction on letter height and line weight.

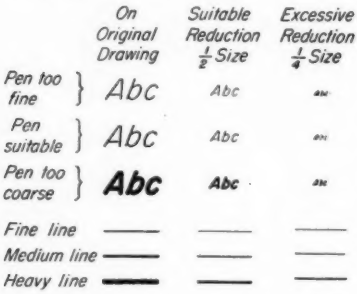


Fig. 1—Effect of size reduction on letter height and line weight

Line weights and letter heights

Often the effectiveness of a chart will depend upon the use of good judgment in the choice of line weights and letter heights The commonly used rectilinear graph is shown in Fig. 2 as an example of poor treatment in this respect. All lines are the same weight and all letter heights the same. The result is a flat, monotonic appearance. Another common error is illustrated in the fact that the area of the graph is much larger than is actually needed for the plotted information.

The same rectilinear chart is made more attractive and interesting in Fig. 3 through several important changes. The range of values is reduced on both the vertical and horizontal axes so that, in each case, the entire scale is utilized in plotting the two curves. The curves are drawn heavier so that they contrast with the light grid lines in the background. Changing the legend from a solid to a broken line for *Sample B* facilitates the reading of the graph. The *X* and *Y* axes at the left and bottom are made slightly bolder than the balance of the grid. Also, the height of the lettering is increased slightly over the height of the numerals in order that the combination of upper and lower case letters will appear to be the same size as the numerals.

*"Mann's Line Gage," published by the Frederick Printing & Stationery Co., St. Louis, Mo.; see Thomas E. French, *Engineering Drawing*, McGraw-Hill Book Co., 7th ed., p. 29.

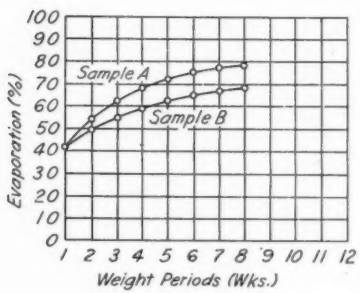


Fig. 2—Rectilinear graph

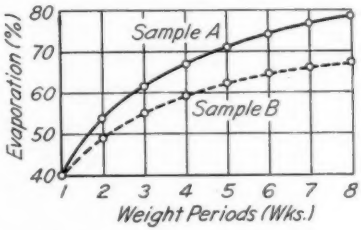
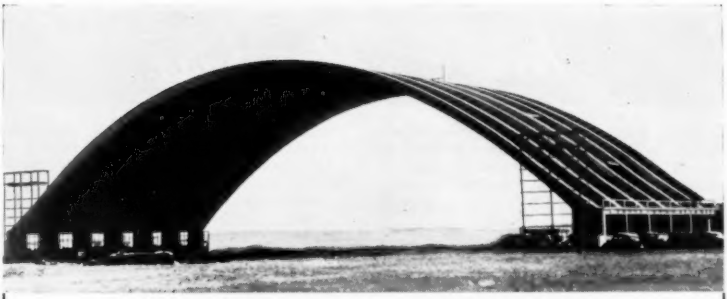


Fig. 3—Rectilinear graph with important changes

Again in Fig. 4 and 5, we have illustrated the value of proper technique in drawing the chart. The drab appearance of the bar chart of Fig. 4 is improved in Fig. 5 by spacing the bars properly, by using heavier lines, by cross hatching to emphasize or accentuate the more important features, and by maintaining a proper letter-numeral size relationship.

Instruments

The rectilinear graph and bar charts also furnish a comparison between hand lettering and instrument lettering. The hand lettering of Fig. 2 and 3 is very flexible since it may be made any size, compressed into a small space, or ex-



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tended to fill a large one. Available to the draftsman are several instruments such as the LeRoy and Wrico sets which utilize templates and a scribe for making letters and numerals. The LeRoy instrument lettering, as shown in Fig. 4 and 5, has the advantage of being more uniform in appearance, but it is not as flexible as hand lettering because the shape of the letter cannot be varied and the size of the letter is restricted by the number of templates available.

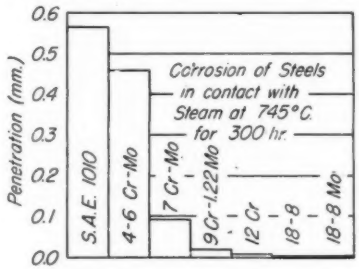

Fig. 4—Bar chart

The factors briefly evaluated in the preceding paragraphs are typical of those encountered in the preparation of graphic materials. Characteristic problems arise with each new and different type of illustration. Observation of accepted practice and application of logical drafting techniques should furnish any author with illustrations which add considerably to the reader's enjoyment of the article.

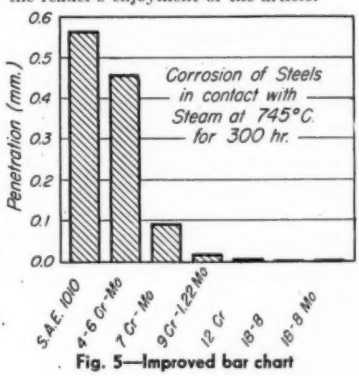

Fig. 5—Improved bar chart

New Members

Sixty applications for membership received in December were approved by the Board of Direction; of these 41 were Individual, 5 Corporation, 5 Junior, 9 Student.

Georgia, New York, Texas and India shared first place honors for the month with credit for four new member applicants each. Pennsylvania, Canada and Venezuela provided three new applicants each for a close second place in this month's standings.

Our membership total January 1, 1950 was 4884.

Colorado

Rose, Edwin, (Indiv.) Denver Federal Center, Building 1-A, Denver, Colo.

District of Columbia

Abul-Fetouh, Abdel-Hadi, (Indiv.) c/o Egyptian Education Bureau, 2200 Kalorama Rd., Washington 8, D. C.

Jelley, J. F., (Indiv.) Chief of the Bureau of Yards & Docks, Navy Dept., Washington 25, D. C.

Florida

Bromilow, Frank, (Indiv.) 406 Reed Laboratory, Dept. of Civil Engr., University of Florida, Gainesville, Fla.

Reynolds, John F., (Indiv.) P. O. Box 4442, Jacksonville 1, Fla.

Georgia

Brown, Earl I. II, (Indiv.) School of Civil Engineering, Georgia Tech, Atlanta, Ga.

Frye, John Spencer, (Indiv.) 100 Waverly Way, N. E., Atlanta, Ga.

Groves, James Leroy, (Indiv.) Star Route, Canton, Ga.

Lose, Robert G., (Indiv.) 1308 Candler Bldg., Atlanta 3, Ga.

Illinois

American Railway Engineering Association, (Corp.) 59 E. Van Buren St., Chicago 5, Ill. (W. R. Wilson)

Newell, Robert H., (Indiv.) 105 Lincoln Place, Ottawa, Ill.

Indiana

Calumet Flexicore Corporation, (Corp.) P. O. Box 356, E. Chicago, Ind (E. B. Mertens)

Iowa

Hansen, Lyle A., (Jr.) 4114 Franklin Ave., Apt. 4, Des Moines, Ia.

Sprinkle, Florian D., (Indiv.) 423 Allen Place Dr., Des Moines, Ia.

Kansas

Lin, Shang-wu, (St.) Applied Mechanics Dept., Kansas State College, Manhattan, Kans.

Louisiana

Ingram, Ray E., (Indiv.) c/o Bodman & Murrell, Archts., 713 Reymond Bldg., Baton Rouge, La.

Massachusetts

Starin, Philip Nicholas, (St.) 532 Beacon St., Boston 15, Mass.

Mississippi

Coggin, B. G. Jr., (Jr.) Box 49, Nettleton, Miss.

New York

Clay, Wharton, (Indiv.) Secretary-Treasurer, Perlite Institute, 35 W. 53rd St., New York 19, N. Y.

Longhi, Edward J., (Indiv.) c/o Westchester Testing Lab., 513 S. Fulton Ave., Mt. Vernon, N. Y.

Pellegrino, Carl, (St.) 1264 E. 94 St., Brooklyn 12, N. Y.

Woodward, Mark R., (Indiv.) The Master Builders Co., 154 W. Huron St., Buffalo, N. Y.

North Carolina

Brenner, Herbert, (St.) 760 Roslyn Rd., Winston-Salem, N. C.

Ohio

Anthony, W. A., (Indiv.) c/o Yoder Co., 5500 Walworth Ave., Cleveland 2, Ohio

Pennsylvania

Carpenter, J. D., (Indiv.) Gannett Fleming Corddry & Carpenter, Inc., 600 N. Second St., Harrisburg, Pa.

Hughes-Foulkrod Co., (Corp.) 809 Schaff Bldg., 1505 Race St., Philadelphia 2, Pa. (Harold M. Logan)

Rigaumont, Victor A., (Indiv.) 5471 Coral St., Pittsburgh 6, Pa.

South Carolina

Cecil, David William, (Indiv.) P. O. Box 1462, Spartanburg, S. C.

McKoy-Werry Co., Inc., (Corp.) Box 609, Spartanburg, S. C. (F. C. Werry, Jr)

South Dakota

Babb, Benjamin M., (Indiv.) Box 161, Pickstown, S. D.

Texas

Alexander, Randle B., (Indiv.) State Bridge Engineer, Texas Highway Dept., Austin, Texas

Ashworth, A. W., (Indiv.) P. O. Box 584, Austin, Texas

Bovay, H. E., Jr., (Indiv.) Esperson Bldg., Houston 2, Texas

Burchfield, R. W., (Indiv.) 2027 Harold St., Houston, Texas

Utah

Utco Products Co., (Corp.) 203 Continental Bank Bldg., Salt Lake City 1, Utah (H. H. Ellerbeck)

Wilson, E. B., (Indiv.) 310 David Keith Bldg., Salt Lake City, Utah

Virginia

Massie, Edgar F. Jr., (Jr.) Box 198, Lynchburg, Va.

Washington

Drollinger, Warren F., (Indiv.) 1604 W. Francise Ave., Rt. 7, Spokane, Wash.

Frese, Verne, (Indiv.) 7265 E. Marginal Way, Seattle 8, Wash.

Wisconsin

Rouman, John James, (St.) 327 E. Lincoln Ave., Tomahawk, Wis.

Canada

McCleery, Raymond Francis, (Indiv.) 9242 Hudson St., Vancouver, B. C., Canada

Mustard, J. Neil, (Indiv.) Hydro-Electric Power Comm., 620 University Ave., Toronto 2, Ont., Canada

Rinfret, Guy R., (Indiv.) c/o The Shawinigan Engineering Co., Ltd., 600 Dorchester St., W., Montreal 2, Que., Canada

Cuba

Reboredo Perez, Rafael de J., (Indiv.) Prado No. 106, La Havana, Cuba

Finland

Nykanen, Arvo Ensio, (Indiv.) 53 A 3 Mannerheimintie, Helsinki, Finland

France

Boue, Jacques, (Indiv.) Ingenieur en Chef des Travaux Maritimes, Arsenal de Toulon, France

Guatemala

Ballerini, Manlio, (Indiv.) 2° Callejon Hall, No. 2 y Calle Montufar, Tivoli, Guatemala City, Guatemala

Continued on p. 36

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Who's Who This Month

John E. Allen

John E. Allen, author of "Construction of Long-Span Concrete Arch Hangar at Limestone Air Force Base," p. 405, is Chief of the Engineering Division of the New England Division, Corps of Engineers which designed Limestone Air Force base under his supervision. At present he is supervising the initial studies for the erection of a proposed 290-ft concrete arch-type dam in Vermont which will be the first flood control dam in New England of other than the earth-fill type.

A graduate of Worcester Polytechnic Institute in 1915, Mr. Allen was employed by the Stone and Webster Engineering Corp. before entering government service in 1934. He was engaged in studies for Ft. Peck and the Passamaquoddy Tidal Power project, then returned to Boston where he was in charge of investigations and design of flood control projects in Maine, New Hampshire and Massachusetts. During the war period, he was in charge of the design and development of U.S. Army airfields, cantonments, ordnance facilities and fortifications in the New England area having a construction cost of over 300 million dollars.

Rudolgh C. Valore, Jr.

"Volume Changes in Small Concrete Cylinders During Freezing and Thawing," p. 417, is the first paper by Rudolph C. Valore, Jr., National Bureau of Standards, Washington, D. C., to appear in the JOURNAL.

The work was inspired by P. H. Bates, retired chief of the Mineral Products Division of the National Bureau of Standards. It represents an effort to explore the mechanics of freezing and thawing. It has long been Mr. Valore's belief that "studies of fundamental nature may indicate, in the near future, the possibility of predicting durability on the basis of one or two cycles of freezing and thawing, with no less confidence than now attaches to the 'make and break' multi-cycle tests." Mr. Valore plans to translate the type of information gained in the volume change study into methods which may be of more practical value.

Mr. Valore has been employed by the National Bureau of Standards since 1938. He has been in the Structural Engineering Section of the Building Technology Division since 1948, and was a member of the Concreting Materials Section 1939-48. He has been engaged in research on dynamic testing and durability studies of concrete and other materials and on studies of lightweight concretes.

He is the author of several papers, including one describing a new method for measuring the bleeding of cement-water mixtures presented at the June 1949 meeting of A.S.T.M. This method, developed with the collaboration of colleagues at the Bureau, forms the basis of a proposed A.S.T.M "Tentative."

A graduate of North Dakota State Teachers College in 1936, Mr. Valore received an M.A. in Education from New York University in 1938.

He is a member of ACI, A.S.T.M. and is chairman of the Highway Research Board committee on dynamic testing of concrete and member of the committee on durability of concrete—physical reactions.

H. W. Russell

Inspection procedures used by the Illinois Division of Highways are described in "Inspection and Control of Concrete for Highway and Bridge Construction," by H. W. Russell on p. 437.

The paper is scheduled to be presented at the inspection session of the 46th Annual Convention in Chicago, February 20.

Mr. Russell has been Engineer of Materials, Bureau of Materials, Illinois

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Division of Highways, Springfield, Ill., since 1942. In this capacity he is in charge of the inspection and testing of all highway materials used in Illinois, as well as supervising the many research projects undertaken by the Bureau of Materials.

He has been connected with the Bureau for the past 28 years. From 1922 to 1933 Mr. Russell conducted research work, with some inspection and testing of various materials used in highway construction. He acted as first assistant to the Engineer of Materials from 1933 to 1942 and was appointed to his present position in 1942.

Eivind Hognestad and Ivan M. Viest

Eivind Hognestad, special research associate of theoretical and applied mechanics, and Ivan M. Viest, special research graduate assistant of civil engineering, both of the University of Illinois, Urbana, Ill., present "Some Applications of Electric SR-4 Gages in Reinforced Concrete Research" on p. 445.

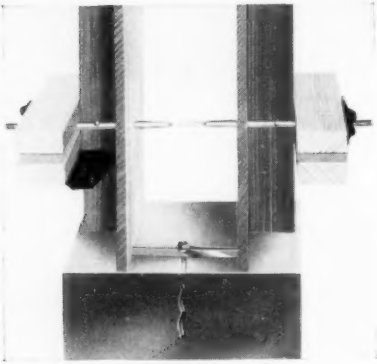
While Mr. Hognestad's first ACI paper is appearing in this issue, he has earlier contributed to discussion, "Letters from Readers" and has reviewed many Scandanavian publications in the "Current Reviews" section of the JOURNAL. He is also the author of a paper on reinforced concrete footings and co-author of a Highway Research Board paper on bond.

In 1947 he received an appointment as a research graduate assistant at the University of Illinois where he worked on various projects pertaining to reinforced concrete research under the supervision of Prof. F. E. Richart and received his M.S. in 1949. He was appointed to his present position in 1949.

Mr. Hognestad graduated from Norway's Institute of Technology in 1947 as a civil engineer after several interruptions of his studies during the German occupation. Following a period as a political prisoner in 1942-43, he escaped to England in 1944 and entered the Royal Norwegian Navy. Returning to the Institute in 1946, he devoted half of his time to concrete research under the supervision of Prof. Inge Lyse while completing his studies. He is a member of ACI and the Norwegian Society of Engineers.

Mr. Viest graduated from the civil engineering department at the Technical University in Bratislava, Czechoslovakia, in 1946, where he also worked for one year as a special assistant in structural engineering. Following a short employment as a construction engineer, Mr. Viest was awarded a scholarship for advanced studies at the Georgia Institute of Technology, where he received his M.S. He was appointed to his present position in 1948, where he has been working on an investigation of reinforced concrete bridges.

Mr. Viest is affiliated with ACI, A.S.C.E., the Society of Czechoslovak Engineers and Sigma Xi.



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S. D. Lash and J. W. Brison

The paper by S. D. Lash and J. W. Brison, "The Ultimate Strength of Reinforced Concrete Beams," p. 457, describes tests made at Queen's University, Kingston, Ont., Canada. The senior author is professor of civil engineering at the University and Mr. Brison was a graduate student research fellowship at the time.

Dr. Lash has been at Queen's University since 1941. He is a graduate of the University of London, England, and was awarded a Ph.D. at the University of Birmingham, England, in 1933 for work on composite steel and concrete construction. He came to Canada in 1929 and has worked for a number of engineering firms and for the National Research Council at Ottawa where he was technical secretary to the National Building Code Committee.

Previous publications by Dr. Lash include a paper on the design of two-way slabs published in the *Engineering Journal* (Engineering Institute of Canada) for which he was awarded the Gzowski Medal. Other papers have appeared in the *Structural Engineer* and in the *Journal of the Institution of Civil Engineers.*

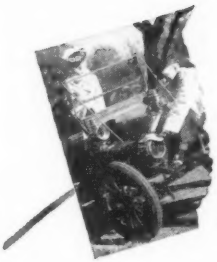
Dr. Lash has been an ACI Member since 1944 and is also a member of the Engineering Institute of Canada, the Institution of Civil Engineers and the Institution of Structural Engineers.

Mr. Brison, a design engineer in the construction design section of the Plant Engineering Dept. of the Ford Motor Co. of Canada, Ltd., is a 1948 graduate of Queen's University. He will receive an M. S. degree from the University in May, which he earned, in part, by his work on the investigation of reinforced concrete beams.

From 1943 to 1945, he served as a navigator in the Royal Canadian Air Force. He is a junior member of the Engineering Institute of Canada.

C. Gray

C. Gray, president of the Ready Mixed Concrete Corp., Indianapolis, Ind., ACI Member since 1940 and past president of the National Ready Mixed Concrete Assn., Washington, D. C., died recently at his home in Indianapolis. Mr. Gray was a graduate of Rose Polytechnic Institute, of which he was a trustee and a member of the board of managers. He was also a lifetime member of Boystown, Nebr. He was chief engineer of the Indiana State Highway Commission from 1920 to 1925 and, during World War I, served as a captain in the Engineers Corps.



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Refer report Committee 804, "Concrete Wearing Surfaces for Floors," Sept. 1938 ACI Journal.

Booklet "Concrete Facts about Concrete Floor Finishes" sent on request.

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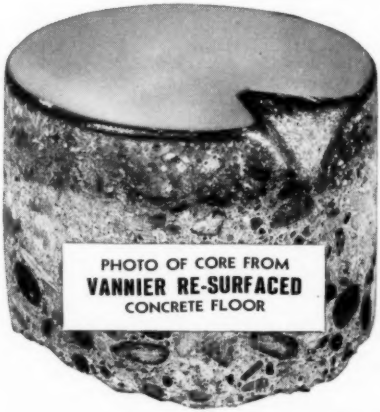

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Honor Roll

February 1, 1949, to December 31, 1949

As the Honor Roll goes to the printer with eleven days left in the current Honor Roll year the Morgans are still "out front". Prof. F. N. Menefee, University of Michigan and Prof. Jose Antonio Vila, University of Cuba are tied for second place honors with 11½ credits each. The final Honor Roll figures will appear in the March Journal.

Newlin D. Morgan (Ill.)	55½
Newlin D. Morgan, Jr. (Wyo.)	27
F. N. Menefee (Mich.)	11½
Jose Antonio Vila (Cuba)	11½
R. H. Sherlock (Mich.)	9
H. J. Gilkey (Ia.)	6½
James A. McCarthy (Ind.)	6
Jaime Alberto Mitrani (Cuba)	6
Howard Simpson (Mass.)	5½
Henry E. Griset (N. C.)	4½
William T. Neelands (Ga.)	4½
F. J. Ochoa U. (Guatemala)	4½
Roberto Barrillas F. (Guatemala)	4
Raymond E. Davis (Calif.)	4
Luis A. Pietri Lavie (Venezuela)	4
J. Vicente Orozco (Mexico)	4
John J. Hogan (N. Y.)	3½
R. R. Kaufman (Ohio)	3½
Ralph W. Kluge (Ind.)	3½
Frank E. Richart (Ill.)	3½
Thomas E. Stanton (Calif.)	3½
H. F. Gonnerman (Ill.)	3
H. D. Jolley (Ill.)	3
Barton Jones (Puerto Rico)	3
J. W. Kelly (Calif.)	3
Frederic F. Mavis (Pa.)	3
Leon Venegas (La.)	3
Stanton Walker (D. C.)	3
J. F. Baxter (N. Y.)	2½
Sterling Lowe Bugg (Fla.)	2½
Harold S. Carter (Utah)	2½
R. A. Caughey (Ia.)	2½
Roger H. Corbetta (N. Y.)	2½
Elmo C. Higginson (Colo.)	2½
A. J. McElrath (Tenn.)	2½
Dan H. Pletta (Va.)	2½
Edward W. Scripture, Jr. (Ohio)	2½
Paul W. Abeles (England)	2
A. Amirikian (Md.)	2
H. J. Bezette (Wash.)	2
W. S. Cottingham (Wis.)	2
Jacob J. Creskoff (Pa.)	2
John G. Dempsey (N. Y.)	2
Ambrosio R. Flores (Calif.)	2
Zorislav Franjetic (Argentina)	2
Robert J. Hansen (Mass.)	2
J. E. Jellick (Calif.)	2
V. P. Jensen (Calif.)	2
Emory E. Johnson (S. D.)	2
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Henry L. Kennedy (Mass.)	2
Karl W. Lemcke (N. Y.)	2
Sam McCluer (Ga.)	2
H. J. McGillivray (Fla.)	2
William S. Moore (Ind.)	2
George G. Nichols (Ore.)	2
William D. Nowlin (Va.)	2
Luis Paredes-Manrique (Colombia)	2
Charles Peterson (Calif.)	2
Wilfrid Schnarr (Canada)	2
Thomas C. Shedd (Ill.)	2
Frederick E. Springate (Calif.)	2
Manabu Tanaka (Hawaii)	2
G. E. Troxell (Calif.)	2
George Winter (N. Y.)	2
John H. Bateman (La.)	1½
S. J. Chamberlin (Ia.)	1½
Leo H. Corning (Ill.)	1½
H. C. Delzell (Ill.)	1½
Peter J. Doanides (S. Africa)	1½
A. H. Douglas (Canada)	1½
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Adli Milik Hanna (Egypt)	1½
Robert L. Mauchel (N. Y.)	1½
Harmon S. Meissner (Colo.)	1½
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Ernest M. Bowles	1
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Carlos Mora Bravo	1
Milton Brumer	1
R. Y. Bush	1
Adraino Carbone	1
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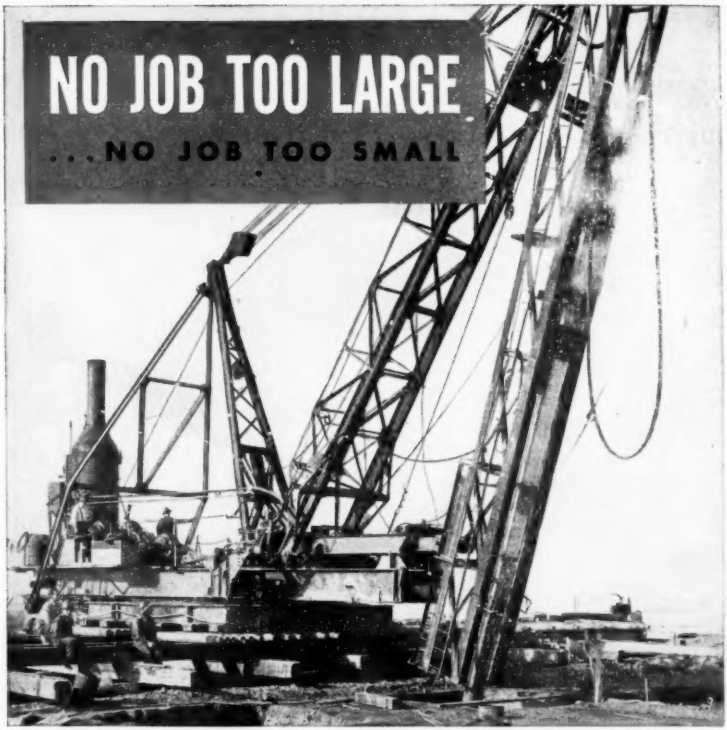
R. V. Chate	1
H. Clare, Jr.	1
Miles N. Clair	1
E. F. J. Clark	1



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W. H. Droege 1
C. Martin Duke 1
Belmon U. Duvall 1
John J. Dwyer 1
Harlan H. Edwards 1
J. M. Faircloth 1
Ewell W. Finley 1
F. L. Fitzpatrick 1
George W. Ford 1
Carl Fox 1
Robert W. Freeman 1
Carmelo Galindez 1
Gerald K. Gillan 1
Castor Segunda Goa 1
Ali N. Gokyigit 1
E. Gonzalez-Rubio 1
Armour T. Granger 1
Samuel J. Green 1
J. E. Guest 1
K. Hajnal-Konyi 1
Robert C. Hanckel 1
George E. Hatch 1
Bernard C. Herring 1
M. D. Hicklin 1
Myle J. Holley, Jr. 1
Percy Howat 1
Franklin D. Howe 1
Myron A. Howe 1
J. W. Hubler 1
Alberto Dovali Jaime 1
J. P. Jain 1
M. E. James 1
V. P. Jensen 1
R. C. Johnson 1
Oliver G. Julian 1
Milton Kellert 1
Frank Kerekes 1
K. V. Kini 1
Reginald A. Kirkpatrick 1
Max Krueger 1
Viljo Kuuskoski 1
Blas A. Lamberti 1
George E. Large 1
William Lerch 1
Harry E. Lewis 1
Leslie D. Long 1
Winthrop E. Luke 1
E. Lyons 1
Charles Macklin 1
Ian MacTaggart 1
Leon C. Magbanua 1
Glenway Maxon 1
D. M. McCain 1
George C. McLeod 1
F. R. McMillan 1
M. J. McMillan 1
Jesus Mesa 1
J. E. Miller 1
Eugene Mirabelli 1
Rafael Garcia Montes 1
Robert B. B. Moorman 1
Robert Morris 1
R. F. Moss 1
Harold H. Munger 1
LeRoy T. Oehler 1
E. L. Oliver 1
Czeslaw Ozieblowski 1
Orin G. Patch 1
Jose F. Perez 1
Anthony F. Pessolano 1
A. Edward Piller 1
J. D. Piper 1
James M. Polatty 1
W. S. Powell 1
Emil Praeger 1
Gayle B. Price 1
Evan L. Richard 1
Raul Miguel Rivero 1
J. E. Robinson 1
Paul Rogers 1
Georges Ronai 1
Albert Rootberg 1
John A. Ruhling 1
Carl E. Sandstedt 1
Emil Schmid 1
Philo H. Schultz 1
F. Selmer 1
Saul Shaw 1
Isaac S. Shina 1
Ranbir Singh Shrikant 1
Richard Skalak 1
Myron M. Smith 1
William Furber Smith 1
Luis G. Solano A. 1
Leroy A. Staples 1
Alois Starkmann 1
Martin Steljes 1
Jiri Stork 1
W. B. Summer 1
E. O. Sweetser 1
Maurice Taylor 1
T. W. Thomas 1
I. L. Tyler 1
Oscar J. Vago 1
Joseph J. Waddell 1
William R. Waugh 1
John M. Wells 1
J. G. Welsh 1
Donald K. Woodin 1

The following credits are, in each instance "50-50" with another member.

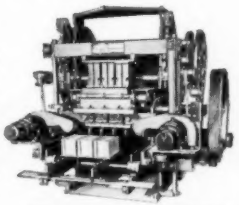
A. L. Alin — Hugh Barnes
Birger Arneberg — Frank Baron
Michel Bakhoum — Frank J. Barrett
C. Harold Barcus — J. F. Barton
Charles H. Barker — J. E. Bates

Continued on p. 40

BESSER VIBRAPAC BLOCK

FOR ALL TYPES OF STRUCTURES

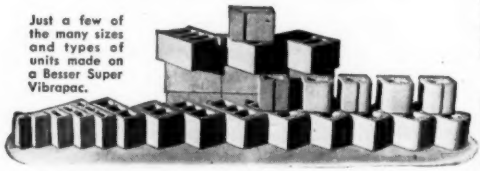
To the BEAUTY and PERMANENCE of Besser Vibrapac Block must be added the VERSATILITY of this modern building material. Its many attractive and distinctive designs, as well as variety of sizes, can be applied equally well to any type of structure. Vibrapac control of block density and texture permit the manufacture of units required for insulation, soundproofing and acoustical control. The durability and design of Besser-made concrete masonry units stand as a building beacon to architects and builders throughout the world.



BESSER SUPER VIBRAPAC — the fully automatic, Plain Pallet block machine. Requires no machine operator. Pallets are fed automatically. Three 8" x 8" x 16" block are made at a time on one Plain Pallet. Smaller sizes in larger multiples on the same pallets.

•

BESSER Vibrapac Better Concrete Masonry Units, in the Hands of Skilled Architects and Builders, Have Become in Effect a New Building Material.



Just a few of the many sizes and types of units made on a Besser Super Vibrapac.

Only ONE set of Plain Pallets required for ALL types and sizes of block. Speeds up block production. Reduces costs.

BESSER MANUFACTURING CO.

Complete Equipment for Concrete Products Plants

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Positions and Projects — ACI Members

As others see us

The following item is reprinted from a column by Thomas Richard Henry in the *Toronto Telegram*, Toronto, Ont., Canada, for December 13, 1949.

Double Talk

In February the American Concrete Institute will hold its annual convention in Chicago.

Looking over the agenda we can't decide whether the convention will be ultra technical—or ultra hilarious.

Among the topics scheduled are: "Structural Design of Joints;" "Deformed Bars and Allowable Stresses;" "Uses of Admixtures to Increase Resistance to Freezing and Thawing;" "Placing of Moment Bars in Precast Joists."

Translations of technical articles

Directory of Translators, Southwestern Research Institute, San Antonio, Texas, managed by Wayne Kalenich, librarian, furnishes (without charge) information about language specialists competent in various subject fields of science and technology. Translators and interpreters may register with the directory.

Mr. Kalenich also manages the Special Libraries Assn. Translations Pool which maintains a record of technical translations (3000 up to date) within private files in this country, and the conditions of their loan or purchase. This pool, created in 1945, can be consulted by those about to undertake a translation and who wish to avoid duplication of work.

PROFESSIONAL CARD

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PRESTRESSED CONCRETE STRUCTURES
Design, Estimates,
Construction Methods, Supervision

Wedding Bells

Although the *News Letter* seldom carries items on weddings, the editors find that congratulations are in order. John L. Savage, 70, well-loved and world famous hydraulic engineer, was married last month to Mrs. James Miner, 60, of Spokane, Wash. Mrs. Miner is the widow of an engineer who worked with Mr. Savage on the Grand Coulee Dam. Mr. Savage's wife died in 1940.

Fraioli, Blum, Yesselman open office

Joseph L. Fraioli, ACI Member, Werner Blum and Joseph B. Yesselman, New York, announced the opening of their structural engineering office in January. The firm will act as consultants on concrete, steel and timber construction, foundations, development and research.

Schroedel elected vice-president of Turner Construction Co.

H. C. Turner, Jr., president of the Turner Construction Co., New York City, announced recently that H. A. Schroedel, operations manager of the Philadelphia office, was elected a vice-president of the company in charge of the Philadelphia office.

Mr. Schroedel is a graduate of Dartmouth College, receiving his C. E. degree in 1926. He has been in the employ of the Turner Co. since the summer of 1926, serving in various junior positions until he became a superintendent in 1935. Prior to his appointment as operations manager in Philadelphia in 1947 he was project manager on some of the largest projects undertaken by the company.

Groll appointed operating manager

Philip A. Groll, who has been manager at the Oglesby, Ill., plant of Lehigh Portland Cement Co., Allentown, Pa., has been appointed assistant operating manager, with headquarters in Allentown.

Just the right Vibrator...

Two CP-220 High-Frequency Electric Vibrators can be powered by a CP-2KW Gasoline-driven Generator

for every concrete job

Seven different models in the complete line of CP Vibrators enable the contractor to select just the type and size for a specific job.

— one man pneumatic and high frequency electric, small diameter models for reinforced concrete, walls and columns, etc.

— one-man and two-man pneumatic and high frequency vibrators for mass concrete.

Latest in the CP line is the light-weight, portable CP-220 high frequency vibrator for concretes of 3″ slump and over. Ask for Bulletin 814.

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How Does Calcium Chloride *Improve* WINTER CONCRETE?

"The Effects of Calcium Chloride on Portland Cement" is a semi-technical book that clearly presents the facts on the use of Calcium Chloride.

SOLVAY Calcium Chloride

√ CUTS DELAYS
√ SPEEDS STRENGTH
√ ADDS EXTRA SAFETY

It is filled with graphs, tables, charts and contains much material not heretofore available. This information will be of great interest to contractors, architects, engineers, plant operators and men in allied fields. For your copy, write on your company letterhead to the address below....there is no obligation.

SOLVAY SALES DIVISION
ALLIED CHEMICAL & DYE CORPORATION
40 Rector St., New York 6, N. Y.

Mr. Groll received his technical education at Rensselaer Polytechnic Institute and graduated in 1921 with the degree of chemical engineer. He started as a chemist at the Alsen plant in 1925, and later worked as a chemist at the Fordwick, Union Bridge and Bath plants. In 1930, he was assigned to the research laboratory at Ormrod and then to Oglesby in 1934 as a chemist and acting superintendent, and subsequently became assistant superintendent and supervising chemist. He was made plant manager in 1945.

Basementless house construction discussed

Basementless house construction was discussed by contractors and builders who attended the fifth annual Short Course in Residential Construction given at the University of Illinois in January under auspices of the University's Small Homes Council. All of the first day's meetings were devoted to this subject.

The second day featured reports on research projects of the Small Homes Council, including the closet-wall, job operations, window and kitchens.

N.S.G.A. and N.R.M.C.A. convention

The National Sand and Gravel Assn. held its 34th annual convention in conjunction with the 20th annual convention of the National Ready Mixed Concrete Assn. in Chicago in January. The program included papers on highways, ready-mixed concrete, aggregates and lightweight concrete.

Gardner named general manager

The South Dakota Cement Commission announced the promotion of Frank Gardner to the position of general manager of the South Dakota Cement Plant, Rapid City, S. D., an ACI Member since 1927.

Are YOU
"MEMBER CONSCIOUS"
? ? ?

"OUTSTANDING ROAD BUILDING UNITS"

'FLEX-PLANE' Joint Installing Machines for installing joints automatically or semi-automatically—we provide electric vibration so the concrete around the joint is thoroughly compacted to prevent cavities, and scaling.

'FLEX-PLANE' Spraying Machines—for curing concrete—an entirely automatic machine—each square yard of surface is double sprayed with proper amount of material.

'FLEX-PLANE' Traffic Line Marking Machines—for marking traffic lanes by use of pigment in plastic concrete.

'FLEX-PLANE' Dowel and Tie Bar Installing Machines —for installing dowels and tie bars in perfect alignment by vibration. No harness required.

'FLEX-PLANE' Finishing Machines—for finishing a complete surface in one pass with a compound screed— 3115 lineal feet of 9″ concrete 24 feet wide in one 11 hour day—a world record!

And Now Our Semi-Automatic Dowel and Tie-Bar Installer

FLEXIBLE ROAD JOINT MACHINE CO.

WARREN, OHIO, U. S. A.

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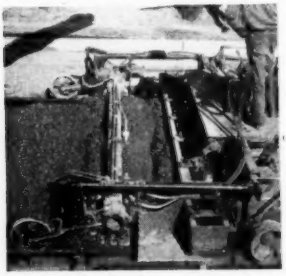
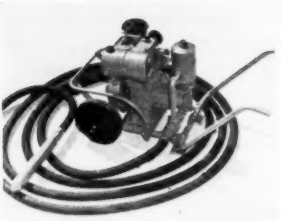
For each and every purpose to which Vibrators are applicable in the concrete industry, we are confident we can supply the equipment that will give you not only the best and fastest placement, but also the maximum of dependability and trouble-free service. Pioneers and outstanding developers of vibratory equipment, our complete line includes internal and external vibrators for: General Construction * Light Construction * Mass Concrete Dam Construction * Hard-to-get-at Places * Form Vibrating * Floors, Streets and Highways * Pipe Manufacturing * Movement of Materials—Vibratory Tables—Thorough soil compaction of bridge approaches, similar fills and concrete floor sub-bases, etc. Drop us a line for complete information.



JACKSON Heavy-Duty Vibratory Paving Tube and Side Form Vibrator

For speedy full width highway and airport concrete paving. Up to 25 feet widths. Submergible dual tubes energized by powerful vibratory motors quickly transform harsh mixes to plastic state. Assures complete compaction — easy finishing. Cement savings up to 10% through reduction in W/C ratio. Attaches to any modern finisher, or rear of spreader for vibrating first course of extra thick slab construction. Variable frequency 3000-5000 V.P.M. Hydraulic lift. Grouped controls.

Our Side Form Vibrator which attaches to any standard finisher eliminates all manual vibrating at side forms and center joints. Rugged, trouble-free.



JACKSON Hydraulic Concrete Vibrator

This machine with its 50 ft. reach saves a great deal of time by making hard-to-get-to places quickly accessible and frequent relocations of the machine unnecessary. Operated by oil pumped through hose to hydraulic motor in vibrator head. Valve on power plant instantly adjusts frequency as desired, from idling to 7200 VPM. Vibrator head is 2¾ diameter. All moving parts run in oil. Hose may be reversed when wear finally occurs. Standard air-cooled gas engine, 7 H.P. A very dependable, powerful and highly popular vibrator.

JACKSON Electric Concrete Vibrator

The new Model FS-150A Electric Flexible Shaft vibrator has powerful 2½ HP Universal Motor; up to 25′ of shaft. Operates from light-socket. 8,000 to 10,000 VPM. Choice of 3 vibrator heads. Quickly converted to wet or dry grinding.

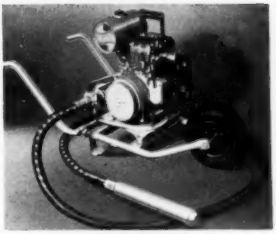
JACKSON Portable Power Plants

The most rugged, reliable plants in existence. Equipped with permanent-magnet generators which require no adjustment or maintenance. Generate both single phase and 3-phase 115 Volt, 60 cycle AC. Capacities: 1.25, 2.5, 5 KVA.

ELECTRIC TAMPER & EQUIPMENT CO.,

The Line that Solves EVERY Concrete Vibrator Problem to BEST Advantage

JACKSON Flexible Shaft Vibrator



Compact and powerful (4 H.P. engine of national renown) built to take severe usage. Available with vibrator heads in 3 sizes and flexible shafting in 7′ and 14′ lengths up to 28′. Full-swivel, dirt-proof base. Furnished with or without wheelbarrow mounting. Frequency is instantly adjustable over a wide range. May be quickly converted to wet or dry grinding and drilling. An excellent vibrator for general construction work. Model FS-6A.

JACKSON Vibratory Soil Compactor

This remarkable new machine will give you up to 95% of maximum density (in granular soils) IN A SINGLE PASS—and do it in one-fifth the time the job may be attempted with other equipment. It is the ideal medium for "presettling" or firmly compacting bridge approaches, similar fills, and sub-bases for concrete floors which are to carry heavy machinery. Easily portable. Propels itself, under normal conditions. Firmly compacts 15 to 18 sq. ft. per minute (granular soil) to a depth of 12″. Time-tested vibratory motor—3-phase, 110 volt, 60 cycle A.C. 4 of these machines may be operated simultaneously by one Jackson (2.5 KVA) power plant.

JACKSON Municipal Paving Unit

Consists of an electric vibratory screed operated by a Jackson Power Plant of 1.25 KVA capacity. Screed is the only hand screed that will undercut at side forms, roll back for second passes, strike off crowns, both regular and inverted, permit operators to work from front, rear or sides. Has such a strong tendency to propel itself in the forward direction that only small effort is necessary to strike off stiff mixes. Manholes and storm sewers are no handicap due to the fact that screed does not reciprocate. Does an excellent job of vibrating concrete to depth of 10 inches on any slab from 6 feet on up to any practicable width. Radically reduces cost and greatly increases production.

LUDINGTON, MICHIGAN

Centennial of the invention of reinforced concrete

The Assn. of Reinforced Concrete Contractors of France (Chambre Syndicate des Constructeurs en Ciment Armé de France) sponsored commemorative ceremonies in Paris, France, in November for the centennial celebration of the invention of reinforced concrete. Twenty nations were represented at the conference.

The program included a review of the history of reinforced concrete and featured papers on the latest design and construction methods. The Museum of Public Works held an exhibition of photographs and models of reinforced concrete structures from early periods to the present. The conference closed with visits to outstanding jobs under construction in the area around Paris.

Laboratorio de Engenharia Civil

The Laboratório de Engenharia Civil, Lisbon, Portugal, was created by a decree of the Ministry of Works in 1946. In creating the new agency, the Portugese government incorporated the Centro de Estudos de Engenharia Civil and the Laboratório de Ensaio e Estudo de Materiais into the new institution. The latter organization was founded in 1898.

It is consulted whenever problems requiring research arise in engineering works conducted by the government or private enterprise in Portugal. The Laboratório also participates in teaching at the national colleges by offering practical courses and student apprenticeship terms which are included in the college curriculums.

Each of three departments is divided into subdivisions. The building research department includes construction methods, standardization and information divisions. The structural engineering department includes divisions on strength of materials, structures, materials, soil mechanics, physics of buildings and hydraulics. The material testing department has two divisions for testing building materials, one handling agglomerates, concrete and natural and artificial stones, the other testing metals and other materials; the third division is devoted to chemical analyses.

Research activities of the Laboratório have included remedies for efflorescence in masonry walls; use of stabilized earth (taipa) in housing; studies of bridge, building and dam models; use of no-fines concrete; effect of sea water on mortars and concrete, use of corrugated iron bars in reinforced concrete, effect of air entrainment and many other investigations.

New Members

Continued from p. 19

Riera, Rafael Perez, (St.) Avenida Hipodromo 43, Guatemala, C. A.

India

Ahuja, P. R., (Indiv.) Central Board of Irrigation, "Kennedy House," Simla 4, India

Bodhe, J. G., (Indiv.) 24-28 Dalal St., Cama Bldg., Fort Bombay, India

De, G., (Indiv.) Director, H. Q., Engineer Research Station, Range Hills Road, Kirkee, Poona 3, India

Gupta, Phool Prakash, (Indiv.) Asst. Engineer, Sarda Power House Construction, Bareilly, U. P., India

Mexico

Lemus Romo, Fernando, (Indiv.) Apartado Postal 220, C. Obregon, Sonora, Mexico

Philippines

Ruales, Rufino J., (Jr.) c/o Cebu Institute of Technology, College of Civil Engineering, Cebu City, Philippines

S. Africa

Stahl, Ludwig, (Indiv.) P. O. Box 8799, Johannesburg, S. Africa

Thorz, Paul, (Indiv.) 84 Commissioner St., P. O. Box 6280, Johannesburg, S. Africa

Switzerland

Daxelhofer, J. P., (Indiv.) Laboratoire d'Essai des Materiaux, Bel-Air Metropole 10, Lausanne, Switzerland

Venezuela

Acosta Sierra, Carlos, (Jr.) Sur 7 No. 149-2 Caracas, Venezuela

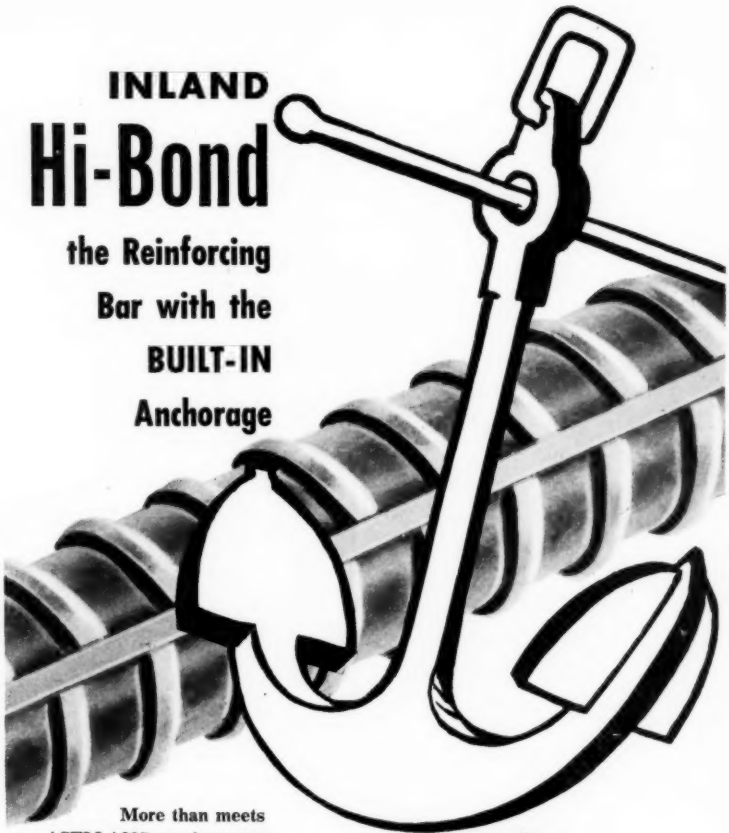
Oliver-Rugelas, Cesar, (St.) Propatria—2a avenida—No. 55, Caracas, Venezuela

Rodriguez Gil, Fernan, (St.) Veroes A. Jesuitas No. 20, Caracas, Venezuela

INLAND

Hi-Bond

the Reinforcing Bar with the BUILT-IN Anchorage



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Other Products:
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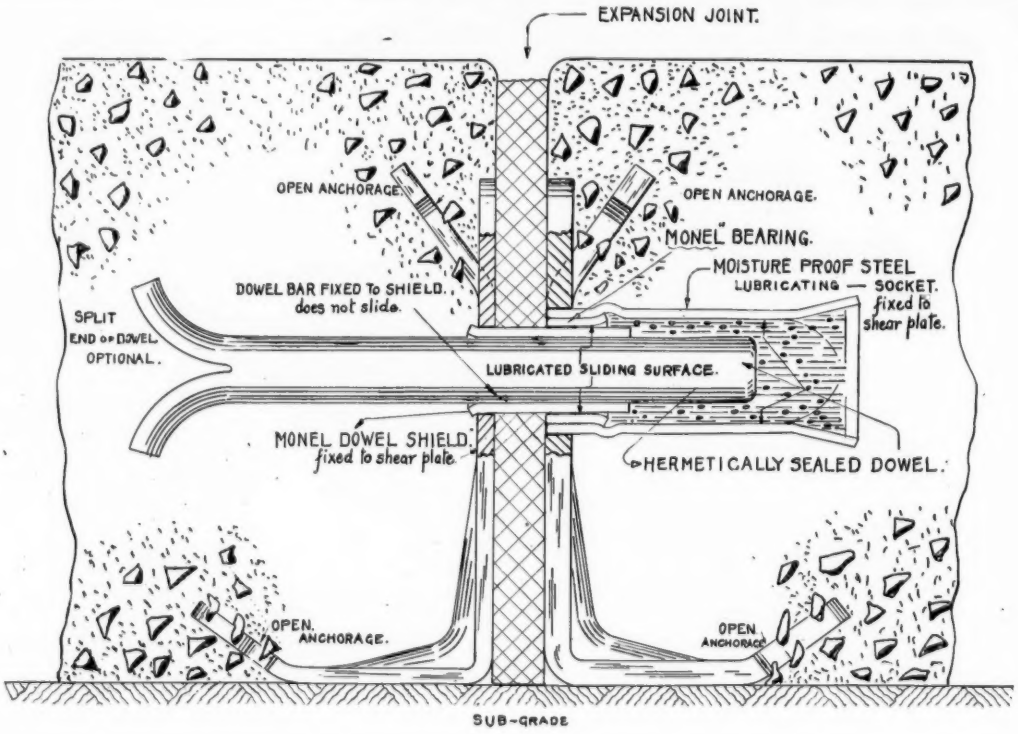
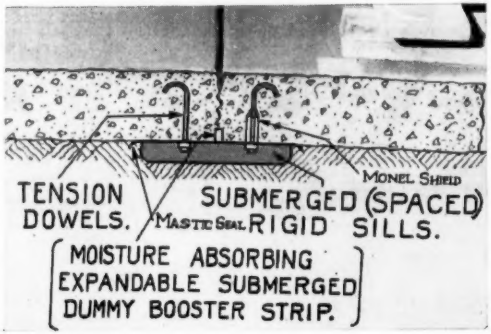
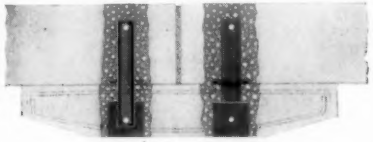

EXPANSION JOINT.
OPEN ANCHORAGE.
OPEN ANCHORAGE.
MONEL BEARING.
MOISTURE PROOF STEEL
LUBRICATING — SOCKET.
fixed to shear plate.
DOWEL BAR FIXED to SHIELD.
does not slide.
SPLIT
END OF DOWEL
OPTIONAL.
LUBRICATED SLIDING SURFACE.
MONEL DOWEL SHIELD.
fixed to shear plate.
HERMETICALLY SEALED DOWEL.
OPEN.
ANCHORAGE.
OPEN.
ANCHORAGE.
SUB-GRADE

Scientists never gave up their determination to produce an Atomic Bomb.



Engineers never gave up their determination to secure a practical load transfer element for expansion and contraction joints for modern concrete highways and airports.

Heltzel engineers have the answer to every single expansion joint problem involving load transfer elements.



Are you interested in permanently lubricated dowels, frictionless dowels—submerged load transfer elements, non-corrosive dowels, hermetically sealed dowels or tension dowels.

Fig. 4—Helco Tension Dowel and Expansion Joint Beam for Highways

HELTZEL STEEL FORM & IRON CO.
WARREN, OHIO, U.S.A.

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Adjustable discharge transit mixer

Bulletin 49-13 published by Chain Belt Co. describes the new Rex Adjustable Discharge Moto-Mixer. Mounting dimensions, drawings and specifications are included in the folder.

For additional information, write Chain Belt Co., 1600 West Bruce Street, Milwaukee 4, Wis.

Iron-Clad concrete floors

A 36-page booklet released by the Master Builders Co. describes the Masterplate Iron-Clad Concrete Floor. The manufacturer states that the Masterplate Floor, with its ⅛-in. iron armored surface, wears 5 to 6 times longer than a plain concrete floor.

Detailed illustrated directions for laying new floors and resurfacing old floors with the new floor material are also given.

A copy of the booklet may be obtained from the Master Builders Co., Cleveland 3, Ohio.

Radiant heating

A bulletin released by the Flexicore Co. shows how to use Flexicore floor slabs in radiant heating systems. The precast slabs provide a "ready-made" panel unit when "radiant" panel heating is specified. Two parallel hollow cores, 4¾ in. in diameter, run the length of each slab, and when the floor is laid, there is one duct to each 6 in. of width.

The bulletin shows how to introduce warm air into these hollow cores to form a panel heating system of the "closed" type. The manufacturer states that the precast unit can also be used for the "split" system ("radiant" panel plus circulating warm air). Warm air flows through the cores, enters the room through a baseboard grill, and returns to the furnace through a conventional cold air duct.

Further information can be obtained from the Flexicore Co., Inc., 1932 E. Monument Ave., Dayton 1, Ohio.

Modular coordination

A 20-page booklet, "Modular Coordination," released by the Housing and Home Finance Agency is the first in a series directed toward getting a wider understanding of what modular coordination means and its benefits. The well-illustrated booklet explains simply what modular coordination is and how it works.

Copies can be obtained from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 15 cents each.

Honor Roll

Continued from p. 28

Aaron H. Bauman
E. W. Bauman
Fred Beatty
H. W. Beckingham
A. H. Brewer
Harold W. Brewer
A. R. Brickler
A. A. Brielmaier
Ralph A. Britson
Robert G. Burnett
Henry B. Campbell
J. A. Carrick
Napoleon V. Campomanes
W. R. Chandler
Arthur P. Clark
Leo Coff
Sidney P. Cohen
T. F. Collier
R. D. Conrow
W. A. Coolidge
Dwight A. Covington
R. W. Coyle
William A. Crabb
Lauro M. Cruz
A. E. Cummings
W. A. Davis
Paul C. Disario
Curzon Dobell
Jay C. Ehle
W. J. Emmons
J. L. Faisant
A. V. Farley
Phil M. Ferguson
Jorge Figuls
Benjamin Forsyth
James J. Fox
Daniel Frederick
J. N. Galli
Ian Glauber
Walter Griesbach
A. W. Haddow
Homer M. Hadley
Stanley P. Haendel
J. R. G. Hanlon
W. S. Hanna
Milton Hartstein
M. J. Hawkins
John M. Hayes
James A. Hedges
James A. Highsmith, Jr.
Frank R. Hinds
A. E. Hiscox
Samuel Hobbs
S. C. Hollister
Albert C. Hooke
S. E. Hyde, Jr.
Melvin Fu Hoon Ja
Frank H. Jackson
Bruce Jameyson
Paul E. Jeffers
Bruce M. Johnson
Samuel Judd
R. G. King
Lane Knight
Carl A Koerner
Fred F. Loy
A. E. Mackney
M. F. Macnaughton
James V. Mandia
Henri Marcus
Hudson Matlock
Denis D. Matthews
J. W. McBurney
Robert B. McCalley, Jr.
H. E. A. McCarthy
J. A. McCrory
T. R. McCullough
E. L. McFalls
Jack Robert Miller
Eric C. Molke
Sylvester Morabito
William Morris
William H. Munse
S. Neatwait
Philip M. Noble
Paul W. Norton
Calvin C. Oleson
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SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 21 are currently available at prices indicated. Please order by title and title number.

RECOMMENDED PRACTICE FOR THE APPLICATION OF PORTLAND CEMENT PAINT TO CONCRETE SURFACES (ACI 616-49) 46-1

Price 50 cents (in special covers).

REPORT of COMMITTEE 616—Sept. 1949, pp. 1-16 (V. 46)

Supersedes 38-30 and 45-18.

This ACI standard establishes recommended practices for appropriate usage, age of concrete, preparation of surface, and the preparation, application and curing of portland cement paint. Three appendixes discuss composition, manufacture and storage, and general characteristics and factors affecting durability.

AN ULTRASONIC METHOD OF STUDYING DETERIORATION AND CRACKING IN CONCRETE STRUCTURES.....46-2

Price 35 cents.

J. R. LESLIE and W. J. CHEESMAN—Sept. 1949, pp. 17-36 (V. 46)

A new method and apparatus for field and laboratory testing of concrete is described. The apparatus called the "Soniscope" was originally designed to detect internal cracks in concrete. It develops pulses of ultrasonic sound in the material and measures the velocity of their transmission through it. This pulse velocity has the unique advantage of being independent of the size or shape of the body under test. Measurements can be made with equal facility in mass concrete, slabs or laboratory specimens.

The existence and extent of internal cracks and the depth of visible surface cracks can be determined by the use of this apparatus.

The velocity has been found, by experiment, to be a reliable measure of the condition of the concrete and is particularly useful in deterioration studies. The dynamic modulus can be calculated from this velocity, and values so obtained are found to agree closely with the results of tests using established methods.

MANUFACTURE OF REINFORCED FOAM CONCRETE ROOF SLABS...46-3

Price 35 cents.

I. T. KOUDRIASHOFF—Sept. 1949, pp. 37-48 (V. 46)

The Russian type of lightweight concrete described used a rosin-glue emulsion to preserve the air voids before the initial set of the cement. Shrinkage was decreased and strength increased through high pressure steam curing. The autoclave treated foam concrete used in the production of precast industrial roof slabs had a unit weight of 47 lb per cu ft and a compressive strength of over 500 psi. The lightweight slabs, used in a load carrying capacity and as insulation, reduced construction time by 50 percent and costs by as much as 20 percent. Test data on roof slabs and production procedures are also described.

SUGGESTIONS ON CONCRETE FLOOR CONSTRUCTION 46-4

Price 35 cents.

ERNST GRUENWALD—Sept. 1949, pp. 49-56 (V. 46)

The relationship between good concrete floors and the proper selection of cement and aggregates is discussed. Data are cited to emphasize the advantage of coarse-aggregate mixes over cement-sand topping for concrete floors.

USE OF AIR-ENTRAINING CONCRETE IN CANAL LINING 46-5

Price 35 cents.

JOSEPH J. WADDELL—Sept. 1949, pp. 57-64 (V. 46)

This paper presents a discussion based on field observations of the use of air entrainment in canal lining concrete. The Bureau of Reclamation made studies of air-entraining agents for use in the irrigation canal lining for the Friant-Kern Canal on the Bureau's Central Valley Project in California. Results to date indicate that appreciable benefits accrue when an air-entraining agent is used in concrete which is placed and compacted by means of a mechanical slip-form. Care is necessary in adjusting concrete mixes to incorporate entrained air because of the sensitivity to mix changes of concrete for slip-form placement.

THE USE OF PORTLAND-POZZOLAN CEMENT BY THE BUREAU OF RECLAMATION..46-6

Price 35 cents.

ROBERT F. BLANKS—Oct. 1949, pp. 89-108 (V. 46)

The Bureau of Reclamation has made extensive studies of pozzolanic materials, and portland-pozzolan cements are now being used in many of the major structures built by the Bureau. The properties of portland-pozzolan cement that are used advantageously in the production of mass concrete are described.

RESISTANCE OF CONCRETE AND PROTECTIVE COATINGS TO FORCES OF CAVITATION.....46-7

Price 35 cents.

WALTER H. PRICE and GEORGE B. WALLACE—Oct. 1949, pp. 109-120 (V. 46)

A machine used for producing cavitation erosion in the laboratory is described and the results of tests made to investigate the effect of mix proportions, surface treatment, and protective coatings on the resistance of concrete to cavitation are discussed. Through proper use of these, the resistance of concrete surfaces to cavitation erosion may be extended three or four times, but even the best concrete will not resist the forces of cavitation for a prolonged period. Heavy rubber coatings bonded well to the surface of the concrete have proved effective.

VACUUM PROCESSES APPLIED TO PRECAST CONCRETE HOUSES...46-8

Price 35 cents.

K. P. BILLNER and BERT M. THORUD—Oct. 1949, pp. 121-128 (V. 46)

The use of vacuum processes in precast concrete construction simplifies the building of fire-resistant, durable structures designed to permit the maximum use of like units. Vacuum processes have thus far been used in one- and two-story houses and one-story industrial or warehouse structures, but further developments should make possible similar benefits for multi-story construction.

The specific vacuum methods utilized are: (1) extracting excess water from freshly placed concrete prior to set, thereby increasing early strength and enabling early handling of units, (2) holding forms in place by vacuum and (3) handling and placing finished and hardened concrete units by cast-in-place closures formed and quickly hardened by vacuum processes.

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NEW TYPE OF CONSISTENCY METER TESTED AT ALLATOONA DAM.....46-9

Price 35 cents.

JAMES M. POLATTY—Oct. 1949, pp. 129-136 (V. 46)

Various mechanical devices have been developed for indicating consistency of concrete since the early days of visual inspection after discharge from the mixer. A new type meter to measure the consistency of concrete while it is being mixed was tested at Allatoona Dam and the operation and results are described.

EFFECT OF MIXING SEQUENCE ON THE PROPERTIES OF CONCRETE....46-10

Price 35 cents.

F. L. FITZPATRICK and W. SERKIN—Oct. 1949, pp. 137-140 (V. 46)

The order in which the constituents of concrete (aggregates, cement and water) are combined in the mixing operation, has a significant effect upon the properties of the concrete as to workability, strength, density, surface finish and absorption. Tests are reported.

BOND OF CONCRETE REINFORCING BARS.....46-11

Price 35 cents.

ARTHUR P. CLARK—Nov. 1949, pp. 161-184 (V. 46)

The tests reported were made to compare the resistance to slip in concrete (bond) of deformed bars when tested in beams and companion pull-out specimens, to secure information on the effects of size of bar, the type of deformations on the bars and the strength of concrete on the bond. The bars were cast in a horizontal position in all test specimens. The variables were depth of concrete under the bar, length of embedment of the bar in the concrete, strength of concrete and diameter of bar. Slip of the bar was measured at the loaded and free ends. Three tests were made with 2 in. of concrete under the bar, with 15 in. of concrete under the bar and with 3-in., 12-in., and 16-in. embedments.

Bond strengths for the beams and the pull-out specimens were affected similarly by changes in the geometry of the bars and the bond test specimens. They were greater when the bars were near the bottom than when they were near the top of the specimens. The highest bond strengths were obtained with bars having deformations conforming to suggested requirements for maximum spacing and minimum height and providing ratios of shearing to bearing areas less than 10, usually less than 6.

PERLITE AGGREGATE: ITS PROPERTIES AND USES.....46-12

Price 35 cents.

J. JOHN BROUK—Nov. 1949, pp. 185-192 (V. 46)

Synthetic expanded volcanic rock, better known as perlite is a fairly recent addition to the lightweight aggregate field. Its use in concrete is governed by weight, gradation, mixing procedure, cellular structure, strength of cell walls, insulating properties, cost, etc. Air entrainment appears to be necessary to give a workable, nonsegregating mix. Perlite aggregate blended with sand can be used in high strength structural concrete and concrete products.

THIN WALL CONCRETE SHIP CONSTRUCTION.....46-13

Price 35 cents.

FRANCIS R. MAC LEAY—Nov. 1949, pp. 193-204 (V. 46)

The development of construction methods for placing thin concrete walls is described. After several attempts using the standard method of pouring between double forms, a new method was developed which permitted the successful construction of ¾-in. and 1½-in. concrete walls. Tests were also conducted to determine the practicability as well as the strength of Gunite as a medium to unite precast units into a monolithic ship. Later these methods were used successfully in the construction of a concrete landing craft for the United States Navy.

SPECIFICATIONS SHOULD BE REALISTIC.....46-14

Price 35 cents.

HARRY F. THOMSON—Nov. 1949, pp. 205-220 (V. 46)

Specifications for concrete in moderate-sized and lesser construction frequently contain provisions which are ambiguous, conflicting in application, or do not fully recognize local materials. Most of these questionable features result from (1) inadequate information regarding the characteristics of concrete, or (2) use of ready-written specifications without revision for conditions or changes in standard requirements.

Among the features discussed are: specifying both method and result; "frozen" specifications; habitual use of "1-2-4", multiple provisions for quality; strength without naming consistency; recognition of local materials; use of compression tests; time of placing; bin-test of cement. Numerous quotations from specifications are given, and suggestions offered for bringing the provisions in line with actual conditions.

INSPECTION AND TESTING OF MATERIALS.....46-15

Price 35 cents.

NICOLAAS T. F. STADTFELD—Dec. 1949, pp. 237-248 (V. 46)

This paper sets forth the care taken in the inspection and testing of 7 million barrels of portland cement used in 4 million cu yd of concrete of great uniformity and every indication of durability after 12 years of observation. It stresses for the concrete the importance of low water solubility, freedom from "laitance," lack of cement burns of the workers, low alkali content, and it shows above all the necessity for manufacturing control of the cement clinker. It describes how premature set of concrete was prevented. It also deals with the inspection, grading, and testing of aggregates. The Board of Water Supply specifications for aggregates are given as well as other factors used in securing good material.

FLEXURE OF CELLULAR SHELLS...46-16

Price 35 cents.

F. E. WOLOSEWICK—Dec. 1949, pp. 249-256 (V. 46

After a brief discussion of the uses of cellular shells and methods availdble for the solution of design problems, a sample problem is set up using the theorem of least work. From charts the moments are determined and the effect of added stiffeners is assessed.

CEMENT PERFORMANCE IN CONCRETE EXPOSED TO SULFATE SOILS.....46-17

Price 35 cents.

L. A. DAHL—Dec. 1949, pp. 257-272 (V. 46)

The Long-Time Study of Cement Performance in Concrete deals with the performance of portland cements in concrete under various conditions of exposure in the field. Among these conditions is exposure to sulfate soils, that is, to the so-called "alkali soils." This part of the investigation is reported in Chapter 5, in which the complete data are given. The present paper describes briefly the work reported in Chapter 5 and the conclusions which have been drawn. Those readers who wish to study the results in greater detail are referred to the more complete report.

EARLY STRENGTH OF CONCRETE AS AFFECTED BY STEAM CURING TEMPERATURES.....46-18

Price 35 cents.

JOSEPH J. SHIDELER and WILBUR H. CHAMBERLIN —Dec. 1949, pp. 273-284 (V. 46)

The testing and results obtained on 990 6 x 12-in. concrete cylinders steam cured for various periods and at temperatures ranging from 100 to 200 F are discussed. Strength results are given for ages ranging from 6 hours to 28 days and strengths of companion specimens moist cured at 70 F are given whenever possible. The information presented was obtained for the Bureau of Reclamation for writing specifications covering the steam curing of precast units such as irrigation pipe.

A SHORTCUT FOR DETERMINING REINFORCEMENT IN REINFORCED CONCRETE.....46-19

Price 35 cents.

V. BOGVAD-CHRISTENSEN—Dec. 1949, pp. 285-292 (V. 46)

This paper presents a graph which gives the complete relationship between moments, thrusts, concrete dimensions, reinforcement and resulting stresses for the reinforced concrete members in question.

PRECAST UNITS FOR SHORT-SPAN BRIDGES.....46-20

Price 35 cents.

ROBERT C. HANCKEL—Jan. 1950, pp. 317-328 (V. 46)

Bridge replacement in Lowell, Mass., where minimum traffic interruption was necessary, led to the adoption of precast reinforced concrete units. Available mobile hoisting equipment limited the maximum weight of sections to about 12 tons which permitted small bridges to be precast as complete structures, while for larger bridges, subassemblies were precast and incorporated into the bridge by a cast-in-place deck. The precasting procedures and construction processes are described and illustrated.

INFLUENCE OF SUBGRADES AND BASES ON DESIGN OF RIGID PAVEMENTS.....46-21

Price 35 cents.

KENNETH B. WOODS—Jan. 1950, pp. 329-348 (V. 46)

This paper has been developed from research data, published reports, and experiences gained by observing the performance of both rigid and flexible pavements—particularly as related to subgrade soil textures and the use of base courses. Structural failures of rigid pavements, caused by large concentrations of exceptionally heavy loads, indicate the need for an evaluation of subgrades and bases in determining the most economical design of rigid slabs.

Indications are that the structural capacity of rigid pavements can be improved by the use of location procedures which utilize the best in topographic position and subgrade soil textures. For inferior situations—in regard to position and soils—the use of base courses must be evaluated against the economy of using slabs of increased thickness, more reinforcing steel, or combinations of the two.

It is concluded that it is not entirely feasible, with the present state of knowledge, to standardize rigid pavement design. Rather, the available data indicate that design practices should be developed by regions in which the subgrade soil, availability and type of base course materials, climatic conditions, and traffic volumes and loads are evaluated.

INSPECTION OF MASS AND RELATED CONCRETE CONSTRUCTION.....46-22

Price 35 cents.

LEWIS H. TUTHILL—Jan. 1950, pp. 349-360 (V. 46)

This paper points out that inspection of any kind can be no more effective than that permitted by the specifications and, particularly, by the established job standard of inspection. No distinction is made between the usual activities of inspection and those associated with concrete control since both have the same objective: a serviceable and presentable structure. Details of requirements and procedure to this end are discussed.

EXPERIMENTAL GROUTING INVESTIGATION FOR CHIEF JOSEPH DAM.....46-23

Price 35 cents.

J. M. WELLS—Jan. 1950, pp. 361-376 (V. 46)

This paper describes the development of a method for the control of seepage into the excavation areas of the right abutment, and through the right abutment, at the Chief Joseph Dam project on the upper Columbia River in the State of Washington.

Laboratory studies are presented on various types of grout mixtures, as well as techniques and procedures for drilling in coarse gravel by jetting methods, procedure for grouting pervious gravels and the efficacy of grouting treatment.

USE OF POZZOLANS IN CONCRETE.....46-24

Price 35 cents.

RAYMOND E. DAVIS—Jan. 1950, pp. 377-384 (V. 46)

After stating the characteristics of pozzolonic materials, the effects of replacing with pozzolans part of the portland cement in concrete are considered briefly. Results of tests with fly ash and superfine diatomite are cited to show the possible use of these materials in the East and Midwest where natural pozzolans of the West are not economically available.

CONSTRUCTION OF LONG-SPAN CONCRETE ARCH HANGAR AT LIMESTONE AIR FORCE BASE.....46-25

Price 35 cents.

JOHN E. ALLEN—Feb. 1950, pp. 405-416 (V. 46)

The reinforced concrete arch hangar at Limestone Air Force Base is noteworthy for its size, being one of the largest of this type yet constructed in this country. The superstructure is composed of a thin reinforced arch shell, shaped like an inverted catenary. Construction procedures are described.

VOLUME CHANGES IN SMALL CONCRETE CYLINDERS DURING FREEZING AND THAWING.....46-26

Price 35 cents.

RUDOLPH C. VALORE, Jr.—Feb. 1950, pp. 417-436 (V. 46)

The volume-temperature behavior of small concrete cylinders was observed, using a new mercury-displacement dilatometer, during freezing and thawing cycles in which the range 40 to −20 F was traversed at various rates. Specimens cast from a mix containing 6 bags of portland cement per cu yd, plain and modified by the addition of an air-entraining agent, were tested in air-dry, vacuum-saturated and partially saturated conditions.

Volume-temperature relationships for air-dry specimens yielded uniform thermal expansion data, but those for vacuum-saturated specimens showed departures in the

Continued on p. 50

DISCUSSION

Discussion closed January 1, 1950

Sept. Jl. '49

Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces (ACI 616-49)—Report of Committee 616

An Ultrasonic Method of Studying Deterioration and Cracking in Concrete Structures—J. R. Leslie and W. J. Cheesman

Manufacture of Reinforced Foam Concrete Roof Slabs—I. T. Koudriashoff

Suggestions on Concrete Floor Construction—Ernst Gruenwald

Use of Air-Entraining Concrete in Canal Lining—Joseph J. Waddell

Discussion closed February 1, 1950

Oct. Jl. '49

The Use of Portland-Pozzolan Cement by the Bureau of Reclamation—Robert F. Blanks

Resistance of Concrete and Protective Coatings to Forces of Cavitation—Walter H. Price and George B. Wallace

Vacuum Processes Applied to Precast Concrete Houses—K. P. Billner and Bert M. Thorud

New Type of Consistency Meter Tested at Allatoona Dam—James M. Polatty

Effect of Mixing Sequence on the Properties of Concrete—F. L. Fitzpatrick and W. Serkin

Discussion closes March 1, 1950

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Bond of Concrete Reinforcing Bars—Arthur P. Clark

Perlite Aggregate: Its Properties and Uses—J. John Brouk

Thin Wall Concrete Ship Construction—Francis R. Mac Leay

Specifications Should Be Realistic—Harry F. Thomson

Discussion closes April 1, 1950

Dec. Jl. '49

Inspection and Testing Materials—Nicolaas T. F. Stadtfeld

Flexure of Cellular Shells—F. E. Wolosewick

Cement Performance in Concrete Exposed to Sulfate Soils—L. A. Dahl

Early Strength of Concrete as Affected by Steam Curing Temperatures—Joseph J. Shideler and Wilbur H. Chamberlin

A Shortcut for Determining Reinforcement in Reinforced Concrete—V. Bogvad-Christensen

Discussion closes May 1, 1950

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Precast Units for Short-Span Bridges—Robert C. Hanckel

Influence of Subgrades and Bases on Design of Rigid Pavements—Kenneth B. Woods

Inspection of Mass and Related Concrete Construction—Lewis H. Tuthill

Experimental Grouting Investigation for Chief Joseph Dam—J. M. Wells

Use of Pozzolans in Concrete—Raymond E. Davis

Discussion closes June 1, 1950

Feb. Jl. '50

Construction of Long-Span Concrete Arch Hangar at Limestone Air Force Base—John E. Allen

Volume Changes in Small Concrete Cylinders During Freezing and Thawing—Rudolph C. Valore, Jr.

Inspection and Control of Concrete for Highway and Bridge Construction—H. W. Russell

Some Applications of Electric SR-4 Gages in Reinforced Concrete Research—Eivind Hognestad and Ivan M. Viest

The Ultimate Strength of Reinforced Concrete Beams—S. D. Lash and J. W. Brison

Current ACI Standards

Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces (ACI 616-49)
16 pages in covers: 50 cents per copy (40 cents to ACI Members)

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218 pages, 6x9 reprinting current ACI Standards: Building Code Requirements for Reinforced Concrete (ACI 318-47); Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46), six recommended practices: Use of Metal Supports for Reinforcement (ACI 319-42); Measuring, Mixing and Placing Concrete (ACI 614-42); Design of Concrete Mixes (ACI 613-44); Construction of Concrete Farm Silos (ACI 714-46); Winter Concreting Methods (ACI 604-48); Application of Cement Paint (ACI 616-49); and two specifications: Concrete Pavements and Bases (ACI 617-44) and Cast Stone (ACI 704-44)—ten ACI Standards in one book, $2.50 per copy—to ACI Members, $1.50. The "Detailing Manual" (ACI 315-48) is available separately only at its established price—see above.

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-Entraining Portland Cements or Air-Entraining Materials Added to Batch at Mixer," and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-Entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944, $1.25 per copy, 75 cents to ACI Members.

Air Entrainment in Concrete—Book 2 (1947)

A 204-page compilation of information on air entrainment published after Book 1 (1944), including: "Field Use of Cement Containing Vinsol Resin", and "Laboratory Studies of Concrete Containing Air-Entraining Admixtures," by Charles E. Wuerpel, "Entrained Air in Concrete," a foreword and 14 short papers presented at the 1946 Convention; and discussion of the symposium, reprinted from ACI JOURNALS for September 1945, Feb., June and Dec. Part 2, 1946. $2.25 per copy, $1.50 to ACI Members.

ACI Manual of Concrete Inspection (1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI Members 75 cents.

"The Joint Committee Report" (1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. 140 pages, price $1.50—to ACI Members $1.00.

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This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI Members.

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Synopses

Continued from p. 45

form of transient expansions during freezing, and residual expansions following thawing of the order of 0.8 and 0.4 percent, respectively, a single slow cycle produced decreases in dynamic E exceeding 60 percent.

Much smaller departures were observed for partially saturated specimens (65 to 85 percent of vacuum-saturation) and included, in addition to transient and residual expansions, secondary effects of freezing termed "shrinkage" and relaxation phenomena. The magnitude of the transient and residual expansions appeared to depend upon the rate of cooling and heating, the degree of saturation of the specimen, and, during the slow cycle, upon the degree of supercooling before freezing.

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THE ULTIMATE STRENGTH OF REINFORCED CONCRETE BEAMS...46-29

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S. D. LASH and J. W. BRISON—Feb. 1950, pp. 457-472 (V. 46)

The results of tests to failure on 57 small reinforced concrete beams are presented, and it is shown that they can be explained satisfactorily on the basis of a simple plastic theory of design. Formulas are proposed for determining the moments of resistance and the critical percentages of reinforcement.

Seldom are construction practices as critically examined as in

A Way to Better Pavement Concrete*

BY F. H. JACKSON†

SYNOPSIS

This paper discusses the performance requirements of concrete pavements from the standpoint, primarily, of the quality of the concrete as a material. It is pointed out that the lack of durability of much of our present day concrete may be due to our methods of construction. The writer believes pavement durability could be significantly improved by using a scientifically proportioned mix of dry consistency, well compacted by vibration or tamping, and with close control over aggregate gradation, in place of the oversanded plastic mixtures which we now use. He does not believe that air entrainment is necessarily the final answer to the problem of surface deterioration but that we should seriously consider overhauling our entire construction practice as well as our present methods of controlling the uniformity of aggregate gradations.

Current practice in the construction of concrete pavements and airport runways in Great Britain and past practice in Germany in the construction of the autobahnen are discussed to show that the placing of harsh, dry mixtures with close control of aggregate gradations is entirely feasible from the construction standpoint.

INTRODUCTION

Most highway engineers will agree that too many of our concrete pavements, particularly the heavy duty pavements of the industrial north central and northeastern states, are not giving the service that we have a right to expect from this relatively expensive type of construction. It is true that both traffic and weather conditions in these areas are unusually severe. The combinations of extremely heavy loads and severe winters with recurrent snow and ice requires just about the best in pavement design and concrete quality that is is possible to produce. The question is—are we getting that best?

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†Member American Concrete Institute, Principal Engineer of Tests, Bureau of Public Roads, Washington, D. C.

The ability of a concrete pavement to function satisfactorily throughout its economic life depends not only upon the adequacy of the design (subgrade support, slab thickness, load transfer at joints, etc.), but also upon the quality of the concrete *per se*. This is not to say that the problem of structural design is unimportant. It is highly important but it is, after all, a separate problem outside the scope of this paper. Moreover, as is well-known, even when a pavement has been designed to meet every requirement as to traffic loading, premature failure of the concrete as a *material*, due to lack of adequate resistance to weathering, frequently occurs. This type of failure usually takes the form of surface scaling or so-called "D-cracking" along joints and structural cracks, or both. The resultant rough surface introduces impact so that traffic loads which could be carried without damage on a smooth surface start pounding on the concrete, causing spalling and raveling at the joints, followed eventually by complete disintegration unless adequate maintenance is provided. This can and does happen regardless of the adequacy of the design if the concrete does not have maximum durability.

DEFECTS IN PRESENT AMERICAN METHODS

Due primarily to high labor costs, developments in mechanization of concrete paving equipment in this country have been largely along the line of increasing the efficiency of operation, first through the substitution of machines for hand methods and, second, through the design of equipment primarily for maximum production rather than maximum quality of product. Whether the mixing, placing and finishing machines now in common use also insure the strongest and most durable concrete is a question and, so far as the writer is concerned, the answer is "No." For example, many of our large paving mixers are not designed to handle the dry mixtures best suited for compaction by vibration. These mixers operate best with an easily handled plastic mixture having a slump of 2 in. or more. Furthermore, this is the only kind of mix which the usual reciprocating screed finishing machine will place properly. Under such conditions the finishing machine exerts no compactive effort whatever, but merely smooths off the concrete which has been distributed on the subgrade by the mechanical spreader. It seems reasonable to assume that concrete so placed cannot possibly be as dense, as strong, or as durable as concrete placed by some procedure which thoroughly compacts the mass into place.

The emphasis on workability, the outgrowth of the desire to produce concrete that can be placed with a minimum of compactive effort and that can be easily finished to our exacting standards of surface smoothness, has led to the more or less general practice of designing paving mixtures with an excess of sand. The result is a heavy, porous mortar

top, sometimes ¼ in. or more in thickness, on the surface of the pavement. This mortar top is an easy prey to frost action or to the effects of salt used for ice removal. It frequently scales off, leaving the familiar rough and unsightly surface.

This scaling problem is really serious, particularly on the most heavily traveled sections of main highways in the vicinity of our large northern cities where it often is necessary to take heroic action to reduce driving hazards under extreme ice conditions. It has become so serious, in fact, that some highway officials are considering the substitution of other pavement types for concrete under these extreme conditions. There is also now some question as to whether air-entraining concrete is the final answer. There is some evidence to indicate that air-entraining concrete will not completely resist the application of raw calcium or sodium chloride although this new type of concrete appears to be effective under conditions where it is possible to apply moderate amounts of salt mixed with sand or cinders, or under conditions where the concrete is subjected to straight freezing and thawing only without application of salt.

So far as resistance to straight freezing and thawing is concerned, the evidence of about 10 years' experience would indicate that air entrainment definitely improves durability. This is shown not only by laboratory test data but by actual field use. The Long-Time Study of Cement Performance in Concrete, initiated about 10 years ago to study the effect of quality of cement upon durability, has demonstrated marked superiority for this type of concrete as compared to the ordinary type. However, we should not assume that merely because we have added 4 or 5 percent air to our present day concrete, we have eliminated all of our troubles and have nothing further to worry about. Nor is merely sealing the surface with a light bituminous material necessarily the final answer, although such procedure undoubtedly helps.

PROPOSED IMPROVEMENTS

In the writer's opinion, the answer to the problem of surface durability of concrete pavements lies in another direction, that is, in the use of aggregate gradations and construction procedures that will make it possible to place denser and more uniform pavement concrete than is possible under present construction practices. To that end, we should develop equipment that will be capable of mixing, distributing and *thoroughly* compacting 0 to ½ in. slump concrete containing a minimum of sand and also make such changes in our specifications for aggregates as will be necessary if we are to use this kind of concrete. The whole construction operation needs overhauling and the following illustrations will show that such proposed changes are not only theo-

retically possible but also entirely practical from a construction standpoint.

EUROPEAN PRACTICE

During the summer of 1947 the author, accompanied by Harold Allen, Chief of the Soils Section of the Bureau of Public Roads, had the opportunity to inspect about 1000 miles of concrete pavement on the German autobahnen as well as a number of concrete pavements and airport runways in England and Wales, including several jobs under construction. A paper on the autobahnen inspection was presented at the annual meeting of the Institute in Denver, and subsequently published in the June 1948 issue of the ACI JOURNAL. In this report, also published in *Public Roads*, various factors were discussed which, in the author's opinion, were responsible for the generally excellent condition of the German roads. It was found that, structurally, the German roads were behaving in much the same way as concrete pavements in this country. This applied particularly to transverse cracking. However, such defects as joint faulting, mud pumping, slab warping, slab settlement and the like, were, in general, much less severe than are frequently found in America. In fact, there was no evidence whatever of the mud pumping at joints which is so prevalent on some of our heavy duty pavements.

The Germans, although following American practice almost completely in the matter of slab design, were not content to follow us in construction procedures. They adopted the general European practice of using a much drier mix with the maximum coarse aggregate size limited to about 1 in. However, they departed from the prevailing European practice of hand tamping or the use of small hand-operated vibrators, by developing very heavy mechanical vibrators and tampers, together with mixers, spreaders and other equipment, all designed to handle very dry concrete. In fact, they carried mechanization to an extreme, just as we do, except that their equipment was designed to develop to the greatest degree the potential strength and density of the concrete rather than maximum production as has been the practice in this country.

A rather full description of the equipment and construction methods used in building the autobahnen is contained in the report previously mentioned. Included in that report are a number of photographs of actual construction operations. These pictures illustrate better than words the dry consistency of the concrete, and also the nature of the equipment used for placing it. They show that is is practical to handle this type of concrete on a large scale production basis. However, the German engineers did not emphasize speed as we so often do. According to available records, their average production rate was in the neighborhood of 1000 ft of 10-in. pavement, 25 ft wide, in 16 hours. Not so fast in terms of current practice in this country.

Experience has shown that this dry, harsh concrete is much more critical with respect to handling, placing and finishing than our usual plastic oversanded mixes. For this reason it is essential that, having once designed a mix for a given project, a high degree of uniformity be maintained throughout construction. This was accomplished by the German engineers by requiring that the aggregate be batched in several separated sizes, their arbitrary separation between fine and coarse being 7 mm (about ¼ in.). The number of required fractions varied from job to job. However, a typical separation might employ six sizes, four of fine and two of coarse aggregate. With so many separations it was possible not only to use an ideal combined grading, but also to maintain a high degree of uniformity from day to day. There is definite need for improvement in our current practice along this line.

The surface appearance of the typical autobahnen pavement after 10 years is quite different from the average American pavement of the same age. In almost all cases the coarse aggregate was found to be just below the surface or actually exposed in more or less degree. Very few instances of excess mortar on the surface were noted. These exposed aggregate surfaces are not at all like the typical scaled surfaces which we encounter, but are ordinarily quite smooth with a terrazzo-like texture, probably the result of slight surface wear. In some instances the surface is slightly rough due to particles of coarse aggregate protruding slightly above the mortar. However, it should be emphasized that the slightly rough surface does not adversely affect the rideability of the pavement. Neither should the exposure of the aggregate have any detrimental effect from the standpoint of durability because of the excellent quality of both concrete and aggregate. There was no evidence to indicate that any great difficulty had been experienced in securing a surface which could be properly finished to a reasonably exact standard of smoothness. There were a few instances where there was evidence of surface grinding apparently to correct initial surface roughness. However, these were the exceptions—not the rule.

Evidence of disintegration due to weathering was found at only one place in the entire system—a point in Bavaria about 20 miles north of the Austrian border, at Salzburg. This also is the only place where any appreciable scaling was found. As usual, it is difficult to run down the reason for this. According to one story, this is one place where the control slipped and defective aggregates were used. This condition was found nowhere else on 1000 miles of roads inspected.

Earlier it was mentioned that, after the inspection in Germany, some concrete road work in the British Isles was observed. The British have about the same ideas regarding consistency and top aggregate size as the Germans, although on their heavy airport runway slabs they use a

top size of 1½ in. They are not so meticulous in the matter of separated sizes, particularly insofar as the fine aggregate is concerned. Their practice is to separate the coarser sizes. For example, at the London Airport the following separated sizes were being used:

1½ in.—¾ in.
¾ in.—⅜ in.
⅜ in.—$\frac{3}{16}$ in.
$\frac{3}{16}$ in.—0

Material smaller than $\frac{3}{16}$ in. is considered as sand. The cement content was about 5.5 sacks per cu yd, the water-cement ratio by weight, 0.49, and the slump ½ in. The British have no use for our wet mixes.

Notes were taken of mixes being dumped on the subgrade at Heathrow (London) Airport. This concrete was about ½ in. slump with the maximum coarse aggregate size of 1½ in. The British were constructing a series of concrete runways 12 in. thick. Concrete of this consistency was being compacted in one course with a surface vibrator of the same general type used in this country, and believe it or not, concrete of uniform density, free from honeycomb, was being consistently obtained, as evidenced by numerous cores which were continually being cut from the pavement for test purposes.

The results secured at Heathrow were not unusual in any sense. The same type of concrete was noted on the Filton Airport near Bristol, where they were building 15 in. runways in one course, and also on the Jersey marine road near Swansea, in Wales. These projects illustrate definitely that it is feasible to compact low-slump concrete by vibration through an appreciable depth.

The surface characteristics of the British pavements resemble those of the autobahnen. Nowhere was there evidence of the heavy mortar top seen so often in this country. There is nothing to scale off—consequently they have little trouble along this line. Many pavements near London, including the numerous by-pass roads built to facilitate traffic around the city were examined and found to be generally of excellent quality, at least insofar as could be judged by appearances. A number of old concrete roads to the southwest and north of London, including some very old city streets laid about 1912 were also inspected and found, in general, in good condition.

RECOMMENDED CHANGES

1. It is believed that our present methods of building concrete roads are wrong. The trend for years has been entirely too much in the direction of increasing so-called efficiency of operation—with presumably lower costs—and too little in the direction of improving the quality

of the end product. There would be no quarrel with this philosophy provided the quality of the end product was satisfactory. However, it is becoming all too evident that in many states our concrete pavements are not giving satisfactory service, particularly as regards resistance to surface deterioration, and that, if this type of construction is to retain its place in the sun, something will have to be done about it.

2. There is evidence to indicate that air entrainment as we now visualize it is not the final answer, particularly under conditions where heavy and frequent applications of raw salts are permitted.

3. It is believed that, by making adjustments in materials, equipment and construction procedures, the durability and, incidentally, the strength of pavement concrete can be definitely improved. These adjustments would include:

(a) The substitution of effective vibratory methods for consolidating the concrete for the present method of surface screeding which provides no compaction whatever.

(b) The use of a 0 to ½-in. slump concrete, designed to be placed by vibration in place of the 2 to 3-in. or higher slump concrete now used.

(c) Reduction of the maximum size of coarse aggregate to 1 in. with the following size separations (or others to accomplish the same purpose):

1 in.—½ in.
½ in.—No. 4
No. 4—No. 16
No. 16—No. 50
No. 50—0

These separations ignore the arbitrary distinction between fine and coarse aggregate. After all, why make any distinction? It is the *total* aggregate grading in which we are interested, and the writer is convinced that the improved uniformity which would result from the use of several separated sizes such as these, carefully blended to secure uniform concrete day after day, would make us wonder why we were ever content to use the present haphazard system.

(d) The redesigning by equipment manufacturers of the construction equipment used for mixing, transporting, spreading and consolidating pavement concrete so as to properly handle the drier mix. Assurance has been given that this can be done provided only that we really want "no slump" concrete. It is also reasonably certain that, provided we had such equipment, methods could be developed which would enable us to build concrete roads at little or no additional cost over the methods now in use.

CONCLUSION

To those who may be inclined to question these recommendations, the writer can only say that desperate situations require desperate remedies. A concrete pavement is supposed to represent one of the highest types of road which it is possible to build. According to present prices, a concrete pavement in New York State would cost approximately $4.00 per sq yd in place, exclusive of reinforcing, or about $100,000 per mile for an average 4-lane highway. The taxpayers have every reason to expect years of useful life from such a pavement. To have serious surface scaling and other defects develop before it is 2 or 3 years old is bad. To permit such a situation to develop simply because we are afraid to change our time-honored ways of doing things is not playing fair with those who pay for and use the roads. Why not do something about it?

Discussion of a paper by F. H. Jackson:

A Way to Better Pavement Concrete*

By HENRY COMACK, J. M. FISHER, INGE LYSE, PHILLIP L. MELVILLE, F. N. SPARKES, M. SPINDEL, C. E. WUERPEL and AUTHOR

By HENRY COMACK†

In spite of the apparent success of air-entraining admixtures for more durable pavements, the author has ably presented the necessity of improving construction methods and mix consistency used in this country for pavements. The writer brought up the potentialities and advantages of dry tamped concrete in the discussion of a paper by Morgan Klock in the June 1949 JOURNAL.‡ The cost of equipment has been one of the major objections to the use of dry tamped concrete. Yet, the economy of cement, not vividly brought out in Jackson's paper may offset a significant part of the cost factor. Briefly, the advantages of properly handled dry tamped concrete are: material economy, density, compressive strength, imperviousness to salt solution, and last but not least—durability.

Investigations of the properties of dry tamped mixes and field equipment for compaction have shown that some types of mixers used for asphalt may be adapted to dry tamped concrete; due to its dry consistency it may be transported from a central mixer by dump truck.

It is the writer's suggestion that ACI form a special committee, investigate what has been done in this field and determine how practical studies of dry tamped concrete for pavements may be initiated. Educators, private and governmental laboratories, state and county highway departments, industrial research associations and equipment manufacturers should participate in an experimental project culminating in the construction of a test highway subject to extremes of weather conditions, and carrying a large volume of traffic.

*ACI JOURNAL, Mar. 1950, *Proc.* V. 45, p. 489. Disc. 46-30 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 4, Dec. 1950, Part 2, *Proceedings* V. 46.

†Flushing, New York.

‡Discussion of "Monolithic and Bonded Floor Finishes," ACI JOURNAL, Dec. 1949, Part 2, *Proc.* V. 45, p. 732-1.

By J. M. FISHER*

The author's enthusiasm for what road builders in Great Britain and on the European continent consider to be good practice will lead ultimately to his goal of better pavements.

No strict comparison between the performance of roads in England and the United States can be drawn because of the great difference in traffic intensity, especially in urban areas, the much wider range of temperatures and greater incidence of icing conditions. In general, however, concrete roads in England would seem to have fewer structural defects and less scaling than those in the United States.

In only one respect does a good quality road in the United States appear to perform better that of its counterpart in England—in riding quality. The marked tendency in American road building to eliminate or drastically reduce the number of joints, coupled with the prolonged working of the heavy mortar layer at the surface of concrete to eliminate anything in the nature of an irregularity, leads to the impression of smooth riding. Whether this impression is factual or not is not known, but some direct comparison of riding quality would be interesting.

The author pays tribute to the quality of post-war road and runway construction in the British Isles. The major portion of this work has been carried out with the aid of American equipment imported during the war for use on airfield construction, although there are a number of high performance British machines available. His questioning of the ability of such machines to turn out and handle effectively the type of concrete which he would like to see used in the United States, and which we normally use for pavement work, would therefore seem to be taking American manufacturers to task. Their pavers will effectively turn out 0 to ½-in. slump concrete; their spreaders will spread it and their finishers, as originally designed, will compact and finish it. But when, because American road builders conceive the idea that the use of vibrators greatly impedes progress, they strip them from their finishers, the resultant mechanical screeding machines will certainly not handle "dry" concrete.

Experience during the recent construction of airfields in the British Isles has shown that a concreting unit consisting of central aggregate and cement batching hoppers, a twin-drum 34E paver, paddle spreader and vibratory finishing machine, all American products, together with ancillary equipment, is capable of producing concrete of the quality cited by Jackson at the rate of 85 to 90 cu yd per hour consistently throughout the working day. This rate of progress must compare favorably with American practice, since it is close to the maximum possible output of the paver. Cores cut from the finished work have shown the vibratory equipment to have been effective in compacting the dry concrete to the full depth of the slab—in some cases more than 12 in. The labor requirement for such work does not seem to be materially different from that in the United States.

*Technical Adviser, John Laing and Son Ltd., London, England.

The author sets forth a series of recommendations as being the way to better pavement concrete—recommendations which are based largely on his experiences on the European continent and in England. If these are adopted, there is little doubt that U. S. engineers will have gone a long way towards this goal. But the equipment with which American road builders have been provided can be used much more effectively than it seems to have been in the past. Some improvements are undoubtedly necessary at all stages in the construction work—we in England are developing what we consider superior machines and techniques, and look forward to producing concrete pavements with every desirable quality, at greater speed and lower costs than are possible at the present time.

By INGE LYSE*

The author has raised an important question in connection with concrete pavement construction. The disintegration of concrete due to freezing and thawing and to ice removing methods is of considerable economic significance in all countries where extreme weather conditions prevail. The paper is therefore a most timely contribution on the problem of improving our methods of making better pavements, and it seems to the writer that the proposed improvements are sound and would lead to strong and durable pavements. An effective vibration of a 0 to ½-in. slump concrete is no doubt much superior to ordinary plastic concrete of 2 to 3-in. or higher slump. Separation of all the aggregate into several sizes and recombination of these fractions into an ideal gradation curve will no doubt give improved quality of the concrete.

As technical adviser on concrete to the Norwegian Bureau of Aviation the writer recommended concrete of very dry consistency for the construction of the new runways for Sola Airport, Norway. The consistency of the concrete was maintained between 10 and 15 degrees VB, corresponding to a slump of less than ½ in. A description of the construction of these runways was given in the paper "The Preparation of Concrete for Airport Runways," Inge Lyse, *ASTM Bulletin*, No. 161, Oct. 1949, p. 48.

Because of the severe frost exposure expected at Sola, air-entraining concrete was specified for the runways. The runways extend close to the shore of the North Sea so that in stormy weather the spray of sea water may cover a portion of them. The combination of sea water and frost severely strains the durability of the concrete and necessitated the best of pavement construction. From 3 to 6 percent entrained air was therefore specified for the concrete. This was later changed to 3 to 5 percent.

Because of the dry consistency of the concrete the concrete finisher was a vibrator type. Fig. A shows the type of paving vibrator selected for the job. It is manufactured by the A/B Vibroverken, Stockholm, Sweden, and consists of a rigid frame on which is mounted a screed for leveling off the mass in front of the main vibrator. Behind the screed is the main vibrator of the road finisher, and behind this, two smaller vibrators placed somewhat

*Professor of Reinforced Concrete Structures, Norway Institute of Technology, Trondheim, Norway.

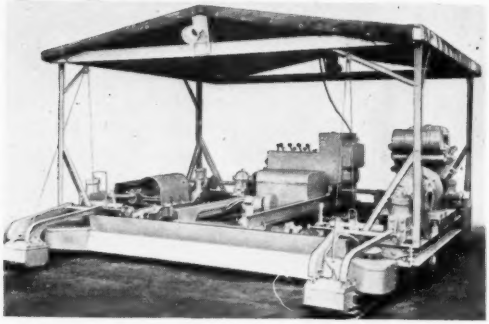

Fig. A—Paving vibrator used for no-slump concrete

diagonally to the direction of the pavement and finally there is a surface finishing screed. The main vibrator thoroughly vibrates the concrete; the diagonally placed vibrators smooth out the surface of the concrete and the finishing screed gives the pavement its final shape. The maximum speed of the road finisher used at Sola was about 3.3 ft per min. The pavement was 15.3 ft wide so that a pavement surface of about 50 sq ft could be laid per min., or 24,000 sq ft per 8-hour shift.

Concrete cores were taken from the pavement to study the penetration of vibration. The pavement was 8 in. thick and all cores showed excellent vibration penetration. Fig. B shows such a core after being subjected to 200 cycles of freezing and thawing. The aggregate is well distributed throughout the section with no indication of surplus mortar at the top. There is no indication of disintegration or surface scaling. It is felt that this method of construction gives durable concrete pavements and should be used wherever disintegration is an important problem.

Only one minor difficulty was experienced in construction. Because of the dry consistency of the concrete and of the proportioning of the mix, some difficulty developed in the final finishing of the pavement surface. There was no excess of mortar or cement paste to give the necessary smooth finish with the simple finishing screed used.

Fig. B—A core from concrete pavement at Sola Airport

It would therefore be an improvement to use a set-up similar to the complete paving equipment manufactured by A/B Vibroverken shown in Fig. C. First comes the concrete spreader which distributes the concrete properly over the prepared subgrade. Then the concrete road finisher thoroughly vibrates the dry concrete. Next the joint machine prepares the joints in the pavement and finally the longitudinal surface finisher gives the final surface smoothness to the concrete. This type of equipment should not be excessively expensive and should give a good concrete pavement for both highways and airports.

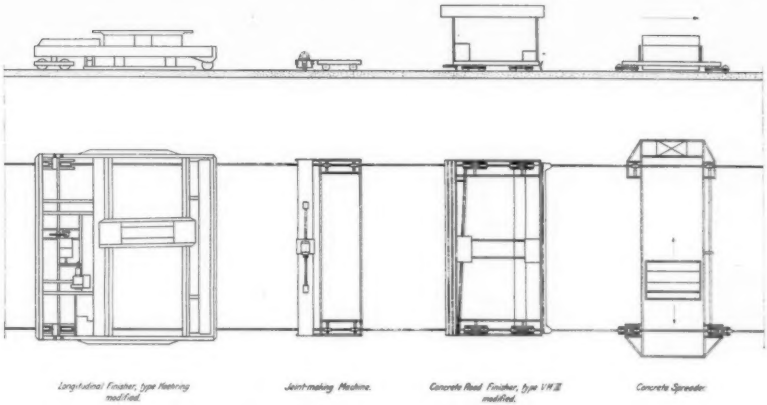
Longitudinal Finisher, type Heltring modified.
Joint-making Machine.
Concrete Road Finisher, type VH II modified.
Concrete Spreader.

Fig. C—Simple paving equipment for no-slump concrete

By PHILLIP L. MELVILLE*

It is indeed comforting to find such a distinguished engineer as Jackson looking critically at concrete pavement construction. It is doubtful if there would ever be any progress if engineers complacently accepted the *status quo*.

The writer was impressed by U. S. construction practices when he came to this country several years ago. Undoubtedly much can be said to the glory of American efficiency and production methods. The world leadership of the United States is a grandiose monument to its industrial and technical achievements.

When he revisited France during the summer of 1949 and inspected a number of construction projects, the writer reflected upon the dilemma of quantity versus quality in concrete construction. Such a dilemma exists at present and it is possible that, in most modern jobs, American practice has been shortsighted in giving quantity too high a priority over quality. It is questionable whether a saving in time is always a good investment and whether a reduction in labor costs will not mean higher total costs with subsequent maintenance and an aborted amortization period. No doubt the situation in Western Europe is extraneous and should not be compared to ours without the utmost care. Cheap labor renders certain practices possible in Europe and, fortunately, the decent level of wages makes them impractical here and requires other solutions from American ingenuity.

As in England, engineers in France use a low-slump concrete. Theoretical considerations (Abrams' and Ferêt's laws) as well as field performance prove that the use of low-slump concrete is sound and desirable when adequately placed. The writer concurs with Jackson that this would require "redesigning by equipment manufacturers of the construction equipment used for

*Associate Research Engineer, Council of Highway Investigation and Research, Charlottesville, Va.

mixing, transporting, spreading and consolidating pavement concrete so as to properly handle a drier mix."

Technical progress should know no geographical boundaries. If the British and the French can design equipment to handle low- (or no-) slump concrete with beneficial results, it is a challenge to the American manufacturer to produce not only machines that would handle these dry mixes but will also handle them efficiently enough to suit our economic requirements.

Another item which may warrant attention is the use of tents to protect fresh concrete from rain, wind, heat and cold. They would in no way impair efficiency and be of high value at a small cost. When the concrete has little or no strength, variations in temperature or moisture during the first 24 hours after placement are most critical. Tents are a cheap and practical means of insulation.

The writer was impressed by the construction of pavements in France and in particular by the care of laborers and engineers to obtain a high quality concrete. The craftsmanship of the type observed at all stages of construction of the West Paris Expressway (*Autoroute de l' Ouest*) will result in a more lasting pavement than might have been expected in this country with current practices. The writer does not believe that European methods should be copied or even imitated but that whenever they are an improvement on ours, principles underlying these improvements should be imported and then adapted to the American scene.

Thus while European contractors and engineers have much to learn from American technological advances, in turn we can at the same time profit from some of the European practices to the benefit of all.

By F. N. SPARKES*

The author has pin-pointed perhaps the only aspect of concrete road construction in which the Americans generally are not leading the world. Admittedly, in comparing American construction procedure with that in Great Britain, Germany and other European countries, it must be remembered that the problems associated with construction in America are different from those on this side of the Atlantic. The enormous distances on the American continent, and the huge road construction schemes associated with them, make it understandable that quality has often been sacrificed to production. Nevertheless, having compared by direct observation the differences between our two countries in relation to the quality of the concrete used, the writer has no doubt that the use of a drier concrete with graded aggregates would not necessarily mean a reduction in the rate of construction, provided that suitable machines for mixing, placing and compacting this type of concrete were made available to the American highway engineer, and would also lead to a considerable reduction, if not elimination, of the scaling troubles to which Jackson refers.

*Dept. of Scientific and Industrial Research, Road Research Laboratory, Harmondsworth, Middlesex, England.

It is common practice in Great Britain on all major highway and runway schemes and even on some of the small highway jobs, such as housing estate roads, to use an aggregate of controlled grading together with weight batching and other aspects of good quality control. As a result, our average mix proportions have changed from the usual 1:6 for hand-laid concrete of about 1 to 2-in. slump to about 1:7½ for mechanically compacted no-slump concrete without loss of strength. As an example of what can be done, given these conditions and adequate control, the details of the mix used on a recent job in South Wales are presented.

Mix proportions by weight: 1 of cement; 0.60 of fine sand; 2.40 of coarse sand; 1.70 of ½ in. crushed limestone; 2.80 of 1½ in. crushed limestone; and a water cement ratio of 0.49. Allowing for the percentage of coarse sand retained on the $\frac{3}{16}$-in. sieve, the quantity of sand ($-\frac{3}{16}$ in.) used was 35 percent of the total aggregate by weight.

This mix, 1:7½ by weight, gave average cube crushing strengths of 5550 and 7310 psi at 7 and 28 days, respectively. On the usual basis that the cube strength corresponds to $\frac{4}{3}$ cylinder strength, the corresponding cylinder compressive stresses become 4200 and 5500 psi, respectively. Cores drilled from this road showed excellent compaction and confirmed that the strength of the concrete as placed was similar to that of the test cubes. The nature of the concrete as placed is shown in Fig. D.

While agreeing with the suggestions made by Jackson, the writer questions the maximum size limit of aggregate. For slab thicknesses of 7 in. and greater we find that it is practical to use a maximum aggregate size of 1½ in. instead of the 1 in. as stated by Jackson. In addition to effecting, in itself, a reduction in the water-cement ratio, it also enables a still lower sand content to be used which effects a further reduction in water-cement ratio.

It is only fair to state that the use of these dry, harsh and lean mixes, while giving a concrete of high strength and durability and low shrinkage characteristics, has introduced new problems, largely associated with the small amounts of new road construction now being built in Great Britain. These include an inferior riding surface, segregation in the concrete, sticking in the concrete mixer and greater difficulty in arrising joint edges; these problems are now being solved. The reasons for the surface irregularities are begin sorted out and recent work suggests that this aspect can be adequately

Fig. D—"Dry" concrete being placed in South Wales, England

controlled. Segregation in the concrete is usually readily corrected by minor adjustments in the sand-gravel ratio while the design of a mixer to mix these dry concretes without sticking is receiving attention. In any case, no sticking is normally experienced when concretes of ¼ to ½ in. slumps are used and it is only with the no-slump concrete that sticking gives some trouble.

It may be argued that the change in construction techniques advocated by Jackson is far too complex to be effected without serious delay while the new technique is being assimilated. Nevertheless, it would be well to construct an experimental stretch of road a few miles long using dry mixes and graded aggregates. If this were laid in line with other stretches constructed both with and without air-entraining agents according to normal American practice and in a state subject to severe winters and to extremes of temperature, the results obtained would be a valuable guide as to whether the real advantages claimed for these mixes on this side of the Atlantic are, in fact, valid under American conditions.

By M. SPINDEL*

There does not yet exist a magic formula for making easily and at low cost, high quality concrete to resist all stresses which can and cannot be foreseen and to have sufficient resistance to severe weather conditions, to percolating aggressive waters and various chemicals and especially to hard wear by traffic and erosion. To design and to make such a concrete has always been a matter of experience and toil.

The satisfactory results obtained in Germany were not due to special enlightenment of their research and practical engineers or to strict orders of German authorities. These achievements were the results of continuous research and practical trials made through the years in a great many countries all over the world. They tried and discussed these problems again and again in their national associations and especially in the International Assn. for Testing Materials, so that the United States, Great Britain and France made their contributions also to the success of the *Autobahnen* in Germany, as did some smaller European countries.

It was not and could not be "a straight line" of improvements but rather "The Zig-Zag Course of Concrete Progress" as pointed out by Herbert J. Gilkey in the April 1950 JOURNAL. It seems that the quality of U. S. concrete pavement is now beneath "the straight line" and to improve it satisfactorily the zig-zag course in the European countries, including Germany, will have to be studied thoroughly.

The author's remark that on some concrete pavements the position "has become so serious, in fact, that some highway officials are considering the substitution of other pavement types for concrete under these extreme conditions" reminds us of a similar position in Europe between 1932-35. At that time most of the experts on large dams had come to the conclusion that

*London, England.

concrete was no longer suitable for large dams and then only if it was faced by natural stone. The writer, who had designed concrete mixes for large dams which withstood water, weather and frost under the severe conditions at an altitude of about 6060 ft, strongly opposed the above conclusion. In a 1935 lecture he pointed out that the failure in many concrete dams was due mainly to the concrete being made on wrong principles—*i.e.*, with too much fine sand, partly increased by grinding of stone, and with too much water content, both used to increase the workability during placement with the equipment then used. Later experience in Europe showed that facing of concrete dams with natural stone had some disadvantages with regard to joints, the masonry work and placing of concrete. Similar difficulties would, no doubt, arise if roads were paved partly with natural stones instead of concrete.

The best remedy was to make high quality concrete and to use not just one of the known methods, but all methods available for making concrete of the highest resistance. For the large dams now under construction at the Tauern power plant, Glockner-Kaprun, in the Austrian Alps near Salzburg, five different sizes of aggregate are stored in bins in addition to those for cement and one for a special admixture for dispersing the cement particles, reducing the water-cement ratio, increasing strengths, density and impermeability, and increasing the resistance to water, weather and frost. This rather dry concrete is placed by vibrating.

The writer agrees that adding 4 or 5 percent air to concrete for pavements cannot eliminate all the troubles and that both the composition and methods of making concrete pavement will have to be altered. The author's suggestions, although useful in principle, will, of course, still have to be worked out in detail.

With regard to air entrainment, in previous discussions of ACI JOURNAL papers the writer pointed out that the advantages and disadvantages of entrained air had to be considered. Those admixtures which disperse the cement particles and reduce the water-cement ratio, if combined with air entrainment, should eliminate or alleviate some of the disadvantages caused by entrained air alone, such as reduced strength, bond and resistance to wear.

In previous discussions the writer also pointed out that it was not sufficient to design the best concrete mixes possible and to control the proportioning, mixing and placing. The composition of the concrete with regard to the absolute volumes of cement, aggregate, water and air and the influence of their mutual ratios had to be dealt with in the four matter parallelogram. In the discussion of "Road Paper No. 17" by F. N. Sparkes and A. F. Smith before the Institution of Civil Engineers, London, in 1945, the authors of that paper agreed that the four matter parallelogram would be a suitable means of testing the efficiency of compaction and of the compacting machines.

The present high standard of road pavements in Great Britain, as described by G. Graham and F. R. Martin in "Heathrow—The Construction of High-Grade Quality Concrete Paving for Modern Transport Aircraft," *Journal*

of the Institution of Civil Engineers, Apr. 1946, is due to the excellent research work by the Department of Scientific and Industrial Research. It is being done by the Building Research Station, as far as the problems of cement and concrete are concerned in general, and by the Road Research Laboratory, dealing with the special problems of roads. Both are collaborating with the Assn. of Cement Manufacturers and with the various committees of the Institution of Civil Engineers and Institution of Structural Engineers.

The author mentioned quite correctly that in England material smaller than $\frac{3}{16}$ in. is considered sand and it is not separated further into different sizes. At Heathrow, however, the grading curve of this sand had to fall within the following limits without being separated.

Passing $\frac{3}{16}$-in.	B. S. sieve between	100 percent and	100 percent
Passing No. 7	B. S. sieve between	70 percent and	90 percent
Passing No. 14	B. S. sieve between	50 percent and	70 percent
Passing No. 25	B. S. sieve between	35 percent and	55 percent
Passing No. 52	B. S. sieve between	10 percent and	30 percent
Passing No. 100	B. S. sieve between	0 percent and	5 percent

As a rule it is not easy to separate sand particles in more than two sizes after passing the $\frac{3}{16}$-in. sieve but, of course, any given limits can be tested by sieving tests as is done for testing the fineness of cements and other powders. The writer has often warned concrete engineers of the damage done to concrete by too high a percentage of fine sand passing the No. 72 B.S. sieve and even more by the fine sand passing the No. 100 B.S. sieve; only low percentages should be allowed.

When the German railways published their book on mortar and concrete about 1927 they forbade all sand particles passing the No. 72 sieve and this, of course, caused serious difficulties. Later, in 1932, the German Ministry limited the percentage of sand particles passing the No. 72 sieve to between 1 and 9 percent for "especially good" concrete and up to 17 percent for ordinary concrete. For the German *Autobahnen* these limits were reduced again to 1-6 percent and 13 percent respectively. This was another example of the zig-zag course of concrete progress with regard to fine sand only.

By C. E. WUERPEL*

The author has presented a splendid and essential critical analysis of the major cause for the growing dissatisfaction with the performance of our modern concrete pavements. Such an analysis is over-due, and serious attention should be given to his appraisal of the situation and to his recommendations for rectification of the faults in construction practice. It is hoped that it will mark the beginning of a definite swing toward modification and improvement of our attitude toward construction as well as to the equipment and materials thereof.

In the paper "An Approach to the Design of Concrete Mixtures," *Civil Engineering*, Aug. 1939, this writer stated that the materials used and their proportioning were of signal importance, but that construction practice,

*Technical Director, Marquette Cement Mfg. Co., Chicago, Ill.

workmanship, was responsible for at least 70 percent of the relative durability of concrete—any concrete. The conviction is held that the direction of recent concern over the durability of modern concrete has been at a tangent to the real truth when reference has been made repeatedly to "old fashioned" cement.

Our basic understanding of the complex physics and chemistry of the hydration process of cement and of the hydrate itself, and our precise knowledge of "old fashioned" cement is inadequate to permit an authoritative appraisal of the broad relative merits of the portland cement of 1920 versus that of 1950. It is doubtful if the answer to that intriguing question will ever be completely available. However, there is ample reason for confidence in our modern cement and also reason for confidence that accelerating *basic* research will provide greatly improved portland cements in the future. At any rate "old fashioned" cement cannot be duplicated for the starkly simple reason that "old fashioned" cement varied through a range of unknown limits of manufacture and physical and chemical properties. Further, progress in the sciences has been and must be the result of improvement of products, improvement in regulation of quality and improvement in knowledge of what "quality" is. We cannot go back!

The foregoing references apply as well to construction practices as to cement. Material factors in the durability of pavements of the past were the time the pavements were cured and held out of service, and the infinitely lighter frequency and weight of traffic imposed, especially in winter. Concrete had full opportunity for development of strength and toughness. We may deplore the speed with which we have conditioned ourselves to move, but can we imagine adopting the old-fashioned practice of keeping a new major traffic artery covered with a deep blanket of wet earth and out of service for 21 or 28 days, as we did in the early twenties due to the slow strength gain of coarse low-lime cements? We cannot, and there is no scientific reason why we should!

However, we must correct the serious faults pointed out by Jackson. The ability of the present-day non-tilting concrete mixer (stationary, transit, or paving) to produce a uniformly homogeneous mass of cement, water, air and aggregate is seriously inadequate—even more so than is implied by Jackson. Workability of mixtures is over-emphasized in the interests of speed and ease. Pavements *are* smoothed rather than compacted in most cases, and finishing of these overly plastic and greatly over-sanded mixtures is over-done.

This writer has advocated low sand-aggregate ratios for years—much lower even than is contemplated usually by that term—and believes this to be an important factor in stable concrete mixtures for *all* types of concrete. He believes, also, that concrete mixtures should be designed for service in the structure rather than for convenience of placement.

Exception is taken to only point in the subject paper: the recommended maximum size of coarse aggregate. A maximum size of only 1 in. in pave-

ments of current thickness will result in more than the optimum quantity of sand and cement (or water) being used. Definite improved quality and stability can be obtained by using *sound* well-graded aggregate up to 1½ or 2 in. in modern pavement and in most structural work. The belief is held, also, that compaction of stiff mixtures in pavements may be achieved better by rolling than by external vibration. The latter, due to the physics of vibration, will accentuate any latent tendency toward over-sandedness at the exposed surface, whereas rolling will not.

This writer would suggest rephrasing Jackson's terminal sentence from "Why not do something about it?" to "We *must* do something about it," if we are to derive from concrete structures the quality which is potential to and inherent in the material.

AUTHOR'S CLOSURE

It is gratifying to note that those who have contributed discussions of this paper have endorsed the general theme; that is, that the quality of pavement concrete in this country should be improved. As a matter of fact, the author, in his contacts with highway engineers throughout the country has encountered few indeed who are completely satisfied with current performance either from the standpoint of structural behavior or durability. The problem is to translate ideas into action. No one seems to be willing to take the first step, primarily because of the prevalent feeling that costs, already high, will go even higher if radical changes in construction methods are adopted.

The author would like to emphasize as he did in the original paper that the problem of structural failure, that is, cracking, joint faulting, mud-pumping, and the like, was not under discussion. The paper had to do entirely with ways and means of improving *durability* of pavement concrete as a *material* and not with structural design problems. It seems important to make this distinction because of the tendency in some quarters to assume that all pavement failures are due to poor concrete and that improvements along the lines indicated in the paper will correct all of our troubles. That this is not true is evident in many places where concrete pavements are breaking up under the action of heavy loads *in spite of* the fact that the concrete is of excellent quality. Such failures are due entirely to overloading and not to any deficiency in the concrete itself. Failures of this type can be prevented only by adequate design plus enforcement of load limitations consistent with the design.

The kind of failure discussed in the paper is independent in large measure of the type or volume of traffic. Surface disintegration (scaling, D-cracks at joints, etc.) as discussed in the paper is the result of weathering action and can only be prevented by improving the resistance of the concrete to this action. The author has suggested certain changes in construction procedures to accomplish this. He repeats the question with which he concluded the paper: "Why not do something about it?"

Noteworthy caisson methods were used in the

Design and Construction of a Circulating Water Intake*

By W. S. COLBY†

SYNOPSIS

The author describes the design and construction of a 250,000 gpm power plant intake. An interesting feature of the pump well cofferdam was the reinforced concrete ring wales, which resulted in savings in operations because the interior of the cofferdam was kept clear for excavating, driving of drilled-in caissons, tremie operations and placing of the pump well concrete.

LOCATION AND GENERAL DESCRIPTION

The Venice power plant is on the east bank of the Mississippi River at Venice, Ill., opposite the city of St. Louis. A general view of the Venice station erected by the Union Electric Power Co. is shown in Fig. 1.

The Venice No. 2 plant was constructed in four sections, starting in 1940. With the installation of two 80,000 kw units of the fourth section now nearing completion, the total installed capacity of this plant will be 400,000 kw. An earlier power plant at the same site, known as Venice No. 1, has an installed capacity of 63,000 kw.

The condensing water requirements of the first three sections of Venice No. 2, as well as Venice No. 1, which have a combined capacity of 303,000 kw, are supplied from intake caisson No. 1. This caisson, originally built in 1925, was extensively revised in 1942 to provide for the installation of four 105,000 gpm pumps which was the maximum pumping capacity that could be installed in the original intake. With the installation of the two 80,000 kw units of the fourth section of Venice No. 2, additional condensing water supply was required. It was therefore decided to install three 83,000 gpm pumps in a new intake system. One of these pumps serves as a spare for the entire Venice No. 2 station.

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†Structural Engineer, Stone and Webster Engineering Corp., Boston, Mass.

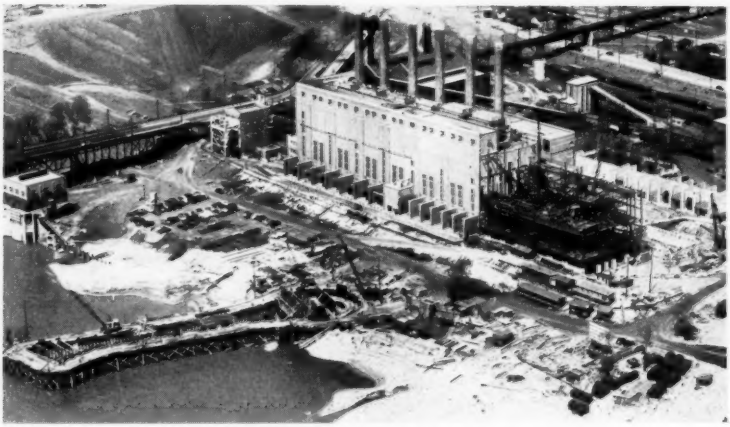

Fig. 1—Aerial view of Venice plant taken during construction of fourth section

The water level in the Mississippi River at Venice varies from an extreme low of gage reading −5.0 to a maximum predicted flood level of gage reading +47.6. The mean water level is at gage reading +12.0 with a normal high water at gage reading +25.0. This wide fluctuation in river level has considerable influence on the arrangement of the circulating water system. The power house is on the land side of the levee system for economy in construction and for protection against flood. The elevation of the power house was determined by the level at which the sum of the capitalized pumping costs and the building costs is a minimum. The pump suctions must be located low enough so that they will be submerged at the lowest river stage. Because of the distance between the power house and the low water line of the river bank, the installation of conduits permitting gravity flow between the river and pump suctions at the power house would prove expensive. In such cases, it has been found economical to locate the circulating pumps, with the protecting trash racks, traveling screens and gates as close as possible to the river bank. This permits the water conduits between the pumps and the condensers to be raised to a more economical level.

The pump wells for both Venice No. 1 and Venice No. 2 are located inside the inner harbor lines to comply with navigation requirements. Water is drawn into the pump wells through intake tubes which extend out far enough into the river to permit full flow at low river stage. It is then raised by vertical pumps and flows through pipes to the condensers and discharges through pipes forming the siphon leg back to the

river. The pump motors, screen drives and control equipment are located in the pump well superstructure above extreme flood level.

SELECTION OF TYPE OF STRUCTURES

Pump well

The pump well for intake No. 1 was constructed using a drop-head caisson. This procedure involved setting up a steel cutting edge on the ground surface and then pouring a concrete shell in horizontal sections on the cutting edge. As the shell was built up the caisson was sunk by excavating the earth inside the shell. This process was continued until the cutting edge was resting on the rock surface. The shell was then sealed, pumped out and the interior walls and water passages constructed. The circular caisson shell was 63 ft outside diameter with concrete walls 4 ft thick. Buried obstructions including water-logged tree trunks caused considerable difficulty in sinking it. When the shell was finally founded on rock, the vertical axis was approximately 8 in. out of plumb. This was corrected in the interior concrete work.

Several considerations raised some doubt as to the practicability of using the drop-head method for pump well No. 2. Borings and construction records indicated that a great many more underground obstructions could be expected at this site. Early maps of the vicinity indicated that at this point a slough had once existed in which sunken barges and old piling were buried deeply in the shifting river silt.

In order that the screen chambers for each pump could be unwatered at any time, the river half of the pump well was divided into three separate sections by division walls. The space required for screens, gates, stop logs and trash racks resulted in an approximately rectangular shape for the intake half of the pump well. The half of the pump well containing the pumps could be made of cylindrical shape.

To withstand the high water pressures which might occur after the pump well was completed or during the sinking of a drop-head caisson of this shape, the intake half of the caisson was designed as a Vierendiel truss with very large moments at the intersections of walls. The interior concrete which would have to be omitted during sinking of the shell and poured later would not be available to resist these moments.

Sinking by the drop-head method would have required excavation from each of the three screen chambers as well as from the pump chamber. It was believed that excavation from the four separate wells combined with the dissymmetry of the structure would add to the difficulty of sinking the shell reasonably vertical.

In structures of this type, which require deep excavation and construction below water, the method of construction may have considerable influence on the design as well as the over-all cost. It is therefore

important that the designers and the constructors work in close cooperation so that provisions may be made in the design to take advantage of construction procedures which may prove economical.

In this instance preliminary bidding plans were prepared on the basis of using a drop-head caisson since it was believed that this method would be the one most familiar to the majority of bidders. Bidders, however, were requested to suggest alternate methods of construction.

The method finally adopted was that proposed by Western Foundation Corp., Spencer, White and Prentis and their subsidiary, the Drilled-In Caisson Corp. who were awarded the construction contract.

In the first step in construction the river bank was excavated to form a level bench at gage reading +11.0.

A construction trestle was then built with its deck at gage reading +30.0. From this trestle a steel sheet pile cofferdam consisting of Z32 piling 75 ft long was driven in a 98 ft diameter circle. Every precaution was taken to ensure driving of the cofferdam in as true a circle as possible. Measurements taken after driving indicated that the maximum variation in diameter was about 6 in.

The earth inside the cofferdam was then excavated to gage reading +4.0 and the upper concrete wale poured. The reinforced concrete ring wales will be discussed in more detail later.

After the upper ring wale had set, excavation and unwatering continued to gage reading −10.0 where the lower concrete ring wale was poured.

Twenty 30 in. diameter drilled-in caissons shown in Fig. 2 were then driven and cored 6 ft into rock by means of a well drilling rig using a 3500 lb drill bit. Steel cores consisting of 14-in. 264-lb H sections were then set and concreted in. The drilled-in caissons were designed to support the downward load of the pump well with eccentricities caused by unwatering various combinations of screen chambers and the earth on the overhanging portions of the tremie mat. They were also designed

Fig. 2—View of drilled-in caissons and trestle for pump well and intake tubes

to resist the uplift on the tremie mat during construction. The caissons were capable of supporting 1100 tons downward load and 550 tons uplift each.

The cofferdam was then pumped full of water so as to balance the water pressure in the river. The earth was excavated subaqueously by clamshell buckets and sand pumps to gage reading −33.0.

A 12 in. thick gravel mat was then placed, prefabricated reinforcing mats and forms lowered into place and the tremie mat poured. Before pouring the tremie mat, the drilled-in caissons were cut off at the proper grade so as to provide support of the tremie mat for downward loads and bond for uplift loads.

The cofferdam was then unwatered, the tremie mat cleaned of laitance and keyways cut in the top of the mat. The construction of the walls and balance of the pump well then proceeded completely in the dry.

The method of construction adopted for the pump well overcame the contemplated difficulties of the drop-head method.

The heavy steel sheet piling of the cofferdam was driven without difficulty through obstructions. Since the bottom of the cofferdam was stopped about 8 ft above the rock surface difficulties of obtaining a seal between the sheeting and the rock surface were avoided. The cofferdam was sealed by the 7 ft 9 in. thick tremie mat with its bottom at gage reading −32 or 21 ft above the rock.

Instead of constructing the interior concrete work to form the water passages and equipment guides as a separate step inside the shell, this method permitted all of the concrete work to be completed in vertical lifts in the dry. The additional interior concrete thus became integral with the shell walls and resulted in a saving of 280 tons of reinforcing and about 13,500 sq ft of forms.

Intake tubes

The intake tubes were built inside a steel sheet cofferdam approximately 250 ft long by 29 ft wide which was driven and laterally supported from two parallel construction trestles. The principal functions of the sheet piling were to permit excavation of the river bank, to confine the tremie concrete surrounding the tubes, and to provide temporary support for the tubes. The interior of the intake tube cofferdam was not unwatered at any time.

Ten 24-in. diameter drilled-in caissons with 12-in. 53-lb cores and 3 ft deep rock sockets and two 30-in. diameter drilled-in caissons with 14-in. 180-lb cores with 5 ft deep rock sockets were driven from the trestles.

After the bottom had been leveled off and an 18 in. thick gravel mat placed, the three 6 ft 6 in. diameter steel intake tubes and a 42 in.

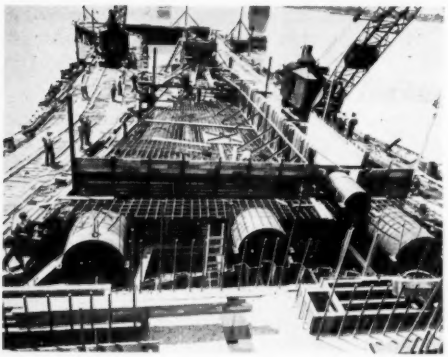

Fig. 3—Construction of intake tubes

warming line were assembled in sections approximately 40 ft long at the top of the cofferdam. The sections were supported by steel frames to keep the tubes rigidly in line. Reinforcing bar cages were also attached to the frames and sheet metal dams were installed at one end of each section to confine the flow of the tremie concrete.

The sections were then lowered and secured in place at the sides of the cofferdam and the surrounding tremie concrete placed one section at a time. Lowering of the sections and pouring of the tremie concrete was started at the river end and progressed back toward the pump well. Fig. 3 shows the next to the last section being lowered into place. Before any section was lowered, the succeeding section was lined up and matched to fit for alignment of pipes. The tremie concrete of any section was not poured until the succeeding section was lowered into place. Split sleeves were used at the ends of the pipes to aid in lining up the pipes and closing the joints.

Short sections of intake tubes cast into the pump well structure were provided with steel bulkheads. After the pump well sheeting was pulled, closure sections were lowered and concreted and the steel diaphragms cut out by divers.

The method adopted for the construction of the intake tubes avoided the use of the heavy cofferdam bracing which would have been required if the cofferdam had been unwatered. The space between the rows of sheet piling was left clear for excavation, lowering the tube sections and tremie operations.

CONCRETE RING WALES

An interesting feature of the pump well cofferdam was the reinforced concrete ring wales. The upper and lower ring wales are shown in

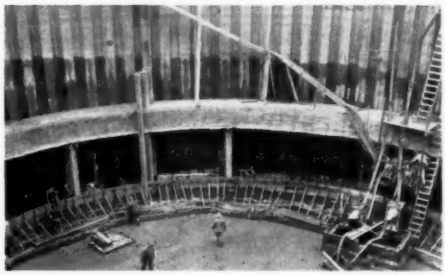

Fig. 4—Ring wales

Fig. 4. It is believed that this was the first use in this country of concrete ring wales for a cofferdam of such large diameter.

The original plan contemplated the use of steel spiders for bracing the sheeting. In addition to the fact that procurement of steel for the spiders would have delayed the progress of work, these spiders would have obstructed subsequent construction operations.

The ring wales were designed to resist water pressure outside the cofferdam with a maximum high water level of gage reading +25.0. Provision was made for flooding the cofferdam when the river stage reached this grade. In addition to the water pressure, the earth was assumed to superimpose a parabolically distributed load on the triangular water pressure diagram.

The maximum radial thrust on the upper wale was found to occur with the cofferdam excavated and unwatered to gage reading −10 in preparation for installing the lower wale. Its magnitude was approximately 40,000 lb per linear ft. The point of contraflexure of the bending moment diagram for the sheeting was assumed to occur 5 ft below the surface of the excavation.

The maximum radial thrust on the lower wale was found to occur after the tremie mat had been poured and the cofferdam unwatered and its magnitude was approximately 46,000 lb per linear ft. If the cofferdam had not been flooded during the excavation for and pouring of the tremie mat, the radial force on the lower wales would have been considerably larger and the steel sheeting overstressed. This would have been due to the increased span of the sheeting from the lower wale to a point of contraflexure below the bottom of the excavation.

If the ring wales were truly circular and subjected to balanced pressures, they would have to be designed for axial compression only as far as earth and water pressures are concerned. It was believed, however, that some consideration should be given to bending moments which could be caused by unbalanced pressures due to possible scouring

action of the river in flood stages, and eccentricities caused by inaccuracies in constructing the wales to a true circle.

To provide for these uncertainties, the following assumptions were made and the wales analyzed by ring analysis.

1. The soil pressure was assumed to be doubled for 30 ft on opposite sides of the wales.
2. The soil pressure was assumed to be doubled for the full width of the cofferdam on opposite sides but passive pressure was assumed to act on the sides at right angles.
3. The wales were assumed to be elliptical in shape with a difference of 4 ft between the minor and major diameters.

The maximum moment was found to occur under the assumption in which the load was doubled for 30 ft on opposite sides of the circle. This moment was about 3,800,000 ft-lb and was combined with a circumferential compression of 2,220,000 lb. Because of the uncertainty of the assumed moment loading conditions and since the cofferdam had been driven so nearly circular, the moment was used only as a guide to determine the amount of reinforcing. Both walers were constructed of 4000 psi concrete with a section 4 ft vertical by 4 ft 6 in. horizontal with 58—1¼ in. square bars placed so as to develop the full section in bending.

The weight of the concrete wales was supported on 12-in. 53-lb steel H piles which were driven in pairs about 30 ft on centers around the circumference of the wales. The steel H piles were driven to rock and the wales poured around the piles to provide support. The outer forms for the wales were placed against the inside of the steel sheet piling. This was done to prevent bonding of the concrete to the piling so as to facilitate later removal of the piling. Provision was also made to wedge between the sheeting and the waler and maintain approximate uniformity of radial pressures if the circle of sheeting became distorted during construction.

It is difficult to estimate the relative economy of the concrete wales as compared with steel spiders. There is no question, however, that the use of the concrete wales resulted in savings in subsequent operations because the interior of the cofferdam was kept clear for excavation, driving of drilled-in caissons, tremie operations and concreting of the pump well structure. The necessity of boxing out around braces projecting through the walls of the pump well was eliminated.

TREMIE CONCRETE

The tremie mat was designed to serve two functions. The first was to act as a seal mat resisting the uplift pressure after the cofferdam was unwatered and before any of the interior concrete work of the pump well itself was started. The second was to act in conjunction with the structural slab poured on top of the mat to resist much higher water pressure in the unwatered portions of the pump well after completion.

The uplift reactions of the seal mat were assumed to be taken by the 20 drilled-in caissons. Eighteen of these caissons were spaced under the walls of the pump well so as to support the gravity loads of the completed structure. Two additional caissons were also located under the interior slab to cut down the span of the mat for uplift forces. It was assumed that no bond would be developed between the edge of the tremie mat and the steel sheeting of the cofferdam. Under these assumptions the maximum moments and shears occurred at the line of the outside row of drilled-in caissons. The mat was designed for a gross water pressure of 3600 psf resulting from a river stage of gage reading +25.0. It was planned to flood the cofferdam if the river stage exceeded this grade.

This design resulted in a slab thickness of 7 ft 9 in. with a reinforcing layer of 1¼ in. square bars placed 15 in. above the bottom of the mat. The tension in the top of the mat caused by moments at interior supports was assumed to be taken by the plain concrete. Top reinforcing was not used because it would have interfered with the placing of the tremie concrete.

In performing the second function of the mat, that is, as the bottom of the completed pump well, the mat was designed to resist pressures resulting from maximum predicted river stage of gage reading +47.6 with a resulting pressure on the bottom of the mat of 5000 psf. The weight of earth backfill outside the pump well walls was assumed to be carried on the projecting portions of the mat as a cantilever. To resist the bending moments caused by this uplift loading, an additional concrete cover slab was placed in the dry on top of the tremie slab. Sufficient reinforcing was placed in this slab to act as reinforcing for the combined tremie mat and top slab.

To develop the longitudinal shear between the tremie mat and the cover slab containing the reinforcing, considerable care was given to the preparation of the top of the tremie mat. A minimum of 3 in. of the top tremie concrete was required to be cut off to insure removal of laitance and unsound concrete. The tremie concrete was placed sufficiently high so that the low spots could be cut the minimum 3 in. without cutting into the designed tremie thickness of 7 ft 9 in. After the top of the tremie was cut down to the proper level the surface was carefully cleaned with a water jet. Immediately before placing new concrete, a 1 to 2 in. coating of grout of the same mix as the subsequent concrete with the coarse aggregate omitted was placed on the cleaned surface. Rather than rely solely upon the bond between the tremie and top covering it was decided to cut keyways in the top of the tremie of sufficient depth and extent to resist the horizontal shear stresses on this plane. These keyways were 4 in. deep by 9 to 12 in. wide.

Because of the use of the tremie mat as a structural slab subjected to large bending moments, it was essential that it be placed so as to form a homogeneous mass without planes of cleavage. Planes of cleavage could have resulted from flow of fresh concrete over previously placed masses which had reached their initial set. This condition could have occurred either from an interruption of supply from the tremie pipes or from trying to supply too large an area from each tremie pipe.

A study of the available sources of concrete indicated that continuous concreting of the 97-ft diameter by 8-ft thick mat requiring 2400 cu yd of concrete without risking interruption of supply was impracticable. The proper distribution of fresh tremie concrete over such a large area would have also required the use of a large number of tremie pipes and a concrete source beyond the limits of the available facilities. It was, therefore, decided to divide the mat into three approximately equal pours of 800 cu yd each by first placing a center section confined between two steel forms. On steel frames light sheet steel piling was mounted with corrugation placed horizontally to form keyways. As shown in Fig. 5 these forms were assembled in sections on the ground with the mats for the bottom reinforcing and then lowered into place under water. Closure between sections and at the periphery of the cofferdam was accomplished by hinged gates which were closed by divers. Additional openings under the forms and at the ends were sealed with sacked concrete.

To determine the proper number of tremie pipes to be used, considerable thought was given to the method of flow of concrete under water. Available information indicated that, in general, flow took place by concrete flowing out over the top of previously placed concrete rather than by forcing the previously placed concrete ahead of it. Under these conditions it was important that succeeding layers of flow should reach

Fig. 5—Tremie forms before lowering into place

the preceding concrete before it had reached its initial set. To study these conditions, two test boxes 5 ft wide, 13 ft 9 in. long and 4 ft deep were filled under water with concrete of the same slump and mix proposed for the tremie mat from a pipe located near one end, using Type I cement in one box and Type III in the other. The flow of concrete was interrupted for periods varying from 1 to 24 hr. Cores were taken through the hardened concrete and tested. The test results indicated that definite planes of cleavage caused by washing out of cement leaving a layer of fine sand and cement laitance occurred whenever the flow was interrupted. The tests indicated surface slopes varying from 1 ft in 6 ft to 1 ft in 10 ft for either type cement but these results were probably affected by resistance at the sides of the test boxes and the scale of the experiment. Measurements taken during placing of the actual tremie mat showed a surface slope of approximately 1 ft in 10 ft.

In the concreting of each third of the mat, five tremie pipes were used. They were spaced about 20 ft apart and 13 ft from each form, and were supported from king post trussed beams located at gage reading +16.0. Just before the conclusion of concreting the tremie pipes were moved slightly toward each form to bring the top of the mat to the proper grade. The appearance of the top of the tremie mat is shown in Fig. 6.

The concrete was proportioned seven bags of high early cement to 1237 lb of sand and 1749 lb of 1½-in. gravel. The slump was about 6 in.

Concrete was fed through the five tremie pipes simultaneously. The pipes were 10¾ in. in diameter, about 40 ft long, with a short section of 18 in. pipe and a steel hopper of about 1 cu yd capacity welded to the top. The tremie hoppers were fed by a pumpcrete machine of 50 to 65 cu yd capacity receiving concrete from transit mix trucks and from two pavers of 40 to 60 cu yd capacity receiving dry batch materials from trucks.

Fig. 6—Top of tremie mat

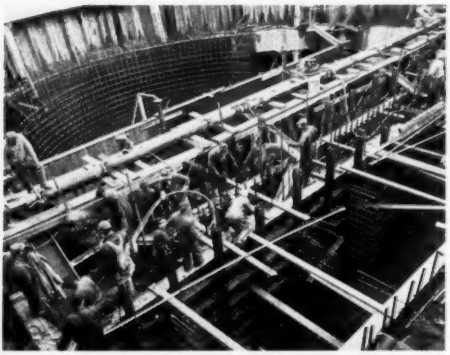

Fig. 7—Interior concrete work in pump well

The middle pour of 800 yd was completed in 16 hours and the outer pours were completed in 6 to 7 hours each.

The cofferdam was unwatered 10 days after the third section of the tremie mat was completed. Some leakage occurred at the joint between the sheet piling and the tremie mat. The leakage amounted to about 10 gpm and was gathered in sumps and pumped out. The river stage did not rise above gage reading +10.00 in the period between the completion of the tremie mat and start of the construction of the pump well walls. It did reach gage reading 25.5, however, shortly after the pump well walls were started. The interior concrete work in the pump well is shown in Fig. 7.

Stone and Webster Engineering Corp. designed the entire Venice No. 2 plant, including the basic design of the pump well and intake, and had general charge of all construction. The detailed structural design and drawings for the pump well and intake were prepared by Charles T. Main, Inc.

Contractor–inspector relationships are discussed from

The Contractor's Viewpoint of Inspection*

By DONALD C. ANDREWS† and NOMER GRAY‡

SYNOPSIS

The interests of owner and contractor are compared and recommendations are made on specifications, personal qualifications of the inspector, and methods of minimizing friction with the contractor.

In the broadest sense, inspection is a control exercised by the owner or his representatives over the materials, methods and workmanship used by the contractor in the performance of his work. The degree of inspection is varied by the type of contract, the kind of work, and the relationships existing between the contracting parties. For this brief discussion, let us confine ourselves to field inspection in the making and placing of concrete. As the major part of such work is done on straight competitive lump-sum contracts, let us assume these conditions.

Obviously, the prime interest of the owner and the contractor are not identical. The owner generally wants the highest quality of materials and workmanship possible under the terms of the contract whereas the contractor tries to perform the specified work as economically as possible.

Few structures are built today with an utter disregard of the economies of construction. Most of our structures are utilitarian and must justify themselves economically. Most owners recognize the importance of economy in construction and try to prepare their plans and specifications so that they indicate the degree of accuracy and the quality of the construction required. Although a few misguided subordinates in engineering organizations may occasionally think only in terms of the highest possible quality attainment, it is improbable that they represent their employers' viewpoint.

Similarly, the contractor, while his prime purpose is to perform the work as economically as possible, has a very strong secondary interest

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†Member American Concrete Institute, Turner Construction Co., New York, N. Y.
‡Member American Concrete Institute, Walter Kidde Constructors, Inc., New York, N. Y.

in seeing that the quality is held to a satisfactory level which will avoid the many pitfalls of slip-shod work. The contractors who would completely subordinate quality to profits are certainly in the minority. Again, a few misguided individuals in contractors' organizations may often think in terms of lowest possible costs with little or no regard for the quality of the structure produced. They no more represent their employers than the misguided inspector represents his.

Plans and specifications vary in clarity, but whether the specifications are brief or voluminous, it is doubtful whether all conditions can be foreseen and provided for in any specification; therefore, an intelligent contractor recognizes the necessity of some field control, and some authority to interpret specifications.

Contractors' supervisory organizations are primarily trained to exhort their employees to greater effort and to perform the work with the least expenditure of materials and labor. It is fairly easy for individual members of the contractor's organization to lose their perspective of the job, and most contractors feel that fair and intelligent inspection will safeguard them from their own errors.

By and large, most contracts are completed with few, if any, serious inspection difficulties. The few cases in which inspection difficulties arise create a furor in the organizations of both parties involved, and are likely to be exaggerated. However, on some contracts, even with the best of intentions by the principals involved, some trouble develops in the field and it is generally blamed upon inspection. In many cases the cause may be over-zealousness of the contractor's supervisory staff, or of the inspector in protecting what he thinks are the best interests of the owner. Experience seems to indicate that most inspection difficulties, stripped down to the essentials, are human rather than technical.

Under the lump-sum form of contract, the scope, quality and manner of performing the work are defined more or less clearly in the plans and specifications. Many of the conflicts, even between well meaning parties, arise from inability to agree on some detail of performance or quality. Clearly then, the more closely contract documents approach perfection, the easier becomes the task of the contractor in preparing his bid and the easier becomes the inspector's task.

Volumes have been written on the subject of specifications. About the best that can be done in this brief treatment is to suggest:

1. That some middle course be adopted in attempting to make them reasonably complete without being voluminous.
2. That requirements pertaining to a particular item of operation be kept together and not scattered throughout the book.
3. That departures from accepted practice be given prominence and not hidden in the middle of some innocuous-looking paragraph.

4. That reference to general standards not printed in the specifications be kept to a minimum.

Our principal point in mentioning specifications is to call attention to the very important role they can play in minimizing the inspector's difficulties and incidentally our own.

The best specifications administered unintelligently will result in trouble, whereas the poorest specifications administered by an intelligent and experienced inspector, generally result in a happy job.

Concrete is the major construction material manufactured at the site. Its manufacture does not lend itself to the degree of standardization and control obtainable in many industrial processes, though we are improving each year. The fact that much of the work is done in the open in all seasons gives rise to situations which, while not new, are frequently not foreseen. This makes inspection work other than simple and routine and calls for experience, judgment and forbearance in an inspector which are unfortunately all too frequently absent.

From the contractor's point of view, far too many inspectors are inexperienced or underpaid. Generally a contractor selects his superintendent only after careful consideration of his experience and his ability to handle men. On the other hand, all too few inspectors are selected by their employers with the same care. Many times the engineer's budget for inspection is too low to afford a really experienced man. Two hazards then present themselves on that job; first, the inspector, due to his lack of experience, is likely to resort to literal interpretation of the specifications, and endless arguments ensue as to what is a straight line or a dry surface; secondly, the underpaid inspector frequently develops resentment because his counterpart in the contractor's organization gets a considerably greater salary than his own, and becomes biased in his decisions, if not actually vindictive.

We think that most responsible contractors ask only that the engineer use as much care in the selection of his inspector or inspectors, as the contractor does in his selection of his supervisory staff. The inspector's position is important, and not one to be relegated to the boss's cousin or that old friend that somebody wants to take care of.

In our judgment an inspector to be effective should have:

1. Experience in the general technique of concrete work and particularly in the type of work in question.
2. An equable temperament and an agreeable manner.

It is probably unnecessary to elaborate on the necessity for experience in an inspector. Unless he can foresee the course of the work in advance, he cannot effectively plan to avoid difficulties and misunderstandings during actual field operations. He clearly cannot foresee job details unless he has been through them before, the more often the better.

No less important is the matter of temperament and manner. We recognize that the inspector has an inherently difficult job. Normally his function is largely negative. He is in the position of remaining silent if an operation is going to his satisfaction and stepping in to restrain when it is not. The manner of conveying his wishes will have an important influence on how they are received and executed.

The contractor's field force is only human. Working under pressure and occasionally danger, they cannot be expected to maintain at all times a sufficiently detached point of view to accept cheerfully what often looks to them like interference by a bystander. An agreeable manner will go a long way toward soothing occasionally ruffled feelings and achieving cooperation.

To get cooperation one must himself cooperate. It will help enormously if the inspector has an understanding of the contractor's problems. This may appear to be asking too much. As the representative of the owner should the inspector be entirely indifferent to the contractor's interest? We believe not.

There are many situations where the inspector can assist the contractor without in any way jeopardizing the owner's interest. Such instances of willingness to cooperate go a long way toward promoting good personal relationships, and the work benefits.

The single factor which probably gives rise to more friction than any other is the sudden, unexpected imposition of an unforeseen requirement by the inspector in the middle of an operation or with such short warning as to upset the contractor's schedule. Field labor costs constituting a large element in construction costs, the contractor strives for a smooth operation and the maximum use of his labor during working hours. Interruption to an operation or upsetting of a schedule costs money.

A great deal of unnecessary friction can be avoided if the inspector will arrange to go over each new operation with the foreman or superintendent in advance. It will then be possible to thresh out any differences at a time when a minimum of interference will be caused. Secondly, it is suggested that the inspector observe preparations for a pour and make his suggestions, if any, before the actual operation begins. Such cooperation will be gratefully received by the intelligent contractor and the work will benefit.

We trust that in making these suggestions we have not implied that a satisfactory contractor-inspector relationship is never found in practice. While perhaps not common, we are optimistic enough to believe that the situation is improving in this regard.

In closing we would say that the "contractor's viewpoint" is only one side of the question. Cooperation is a two-way street.

A new approach to a subject about which little quantitative knowledge exists.

Corrosion Protection of Thin Precast Concrete Sections*

By D. H. PLETTA†, E. F. MASSIE‡ and H. S. ROBINS§

SYNOPSIS

A new electrical resistance technique for measuring the rate of corrosion of steel reinforcing is described. The method employs a thin ribbon 0.008 x 0.25 in. as the resistance element embedded in 6 x 12 in. thin panels, and a Kelvin bridge sensitive to 0.0001 ohm. The data plotted in dimensionless parameter form enable the *half-life* of concrete to be determined at a comparatively early age. The term half-life is defined as the time required for the cross-sectional area of the reinforcing to decrease by one-half its original value due to corrosion. Six mixes, three water-cement ratios, four exposure conditions and three depths of cover were examined.

HISTORY

This paper describes a research project being made for the Bureau of Yards and Docks, U. S. Navy, with funds supplied by it through the Office of Naval Research. The purpose of the investigation is to obtain quantitative data on the amount of protection afforded to reinforcing by thin precast concrete sections. The Navy has been using such construction for warehouses¶ and this development has been under the direction of A. Amirikian, Head Designing Engineer of the Bureau of Yards and Docks and chairman of ACI Committee 324 on Precast Reinforced Concrete Structures.

METHOD

Although the authors reviewed many references published during the past 40 years dealing with corrosion of reinforcing in concrete, they

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†Member American Concrete Institute, Professor and Head, Applied Mechanics Dept., Virginia Polytechnic Institute, Blacksburg, Va.

‡Member American Concrete Institute, Stress Analyst, Montague and Betts, Lynchburg, Va. (Formerly Research Fellow in Applied Mechanics, V.P.I.)

§Research Fellow, Applied Mechanics Dept., V.P.I., Blacksburg, Va.

¶Amirikian, A., "Precast Concrete Storehouses," ACI JOURNAL, June 1947, *Proc.* V. 43, p. 1097.

were unable to find any papers dealing with quantitative measurements of the rate of corrosion. Several methods were considered for determining this data, such as (1) an examination of reinforcing taken from a series of specimens broken at selected time intervals, (2) measuring the change in the electromotive force between the reinforcing and a piece of platinum embedded in the test panel, (3) examination by X-ray, and (4) measuring the change in the electrical resistance of the reinforcing. The last method was adopted as being the most practical. Considerable time was required, however, to eliminate initial failures in technique but the final procedure proved satisfactory.

Test panels, 6 x 12 in., were made in three thicknesses, ¾, 1⅛ and 1½ in. (Fig. 1). A low carbon steel ribbon 0.008 x 0.25 in. was soldered to two ¼ in. square copper bars, and this assembly embedded along the longitudinal centerline of the panel (Fig. 2). Three depths of cover were used, ¼, 7/16 and ⅝ in. The thin flat ribbon was used to increase the resistance and the ratio of surface area to cross section, thus guaranteeing maximum change in cross section and resistance as corrosion progressed.

Companion panels of the same size were also made and these had a ¼ in. round bar 10 in. long embedded in place of the steel ribbon. Companion panels were subjected to the same exposure as the master panels. Both were broken when the resistance measurements indicated complete corrosion of the ribbon, and the ribbon and bar examined visually.

Panels were molded on a vibrating table with an acceleration of 1.7 g and amplitude of 0.01 in. They were cured in the molds one day, in a 70 F, 100 percent humidity room one day and in a 70 F, 50 percent relative humidity room five days before the first resistance reading was taken. The resistance of each ribbon was also measured before embedment. Panels were then subjected to four exposure conditions. Resistance measurements were made periodically in the constant temperature room (70 F, 50 percent relative humidity) after the panels had been stored there for 24 hours. Maintenance of constant temperature is especially important for a 1 F change means a variation of 0.0012 ohm, or 12 times the least reading. A Kelvin bridge with a range of

Fig. 1—Typical panels

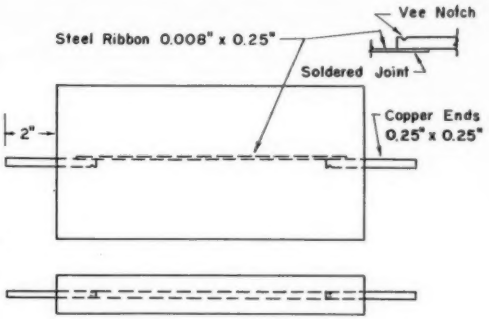
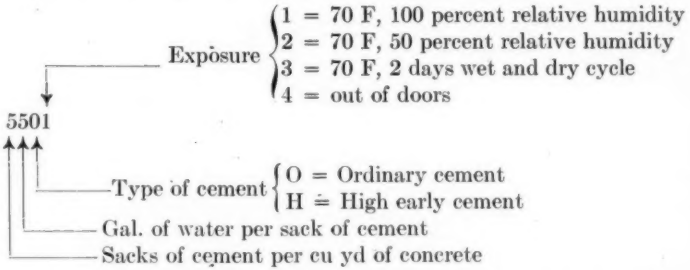


Fig. 2—Details of test panel

0.0001 to 26.6 ohms was used to measure the resistance. Readings were duplicated to ±0.0002 ohms or ±0.1 percent, but only when a double contact key was used. The first set of points closed the battery circuit and the second, closing an instant later, closed the galvanometer circuit. Contact was broken almost immediately to prevent the current from heating the steel ribbon and thus increasing the resistance. Before the double contact key was used, resistance of the ribbon previous to embedment increased with time as the ribbon grew warm. The resistance of the slab measured about 4000 ohms either wet or dry and did not affect resistance readings of the ribbons.

CONCRETE MIXES

Six concrete mixes with three water-cement ratios were used for concretes made of both ordinary and high early strength cements. The numbering system used to identify each mix and the exposure to which it was subjected was of a four digit type illustrated below.


Exposure
1 = 70 F, 100 percent relative humidity
2 = 70 F, 50 percent relative humidity
3 = 70 F, 2 days wet and dry cycle
4 = out of doors
5501
Type of cement
O = Ordinary cement
H = High early cement
Gal. of water per sack of cement
Sacks of cement per cu yd of concrete

The mixes and their plastic ranges are indicated in Table 1. Eight panels were made of each mix, four of the ribbon and four of the companion bar type.

TABLE 1—PLASTIC RANGE OF MIXES

Slump, in.	Gal. water per sack	Sacks cement per cu yd					
		5	6	7	8	9	10
0	5 6	550 Too dry	650	Too dry			
1–3	5 6 7	570	660 670	760	850		
3–6	5 6 7			770	860	950	1050
>6	6 7				Too wet	960 Too wet	1060 Too wet

MATERIALS

Petersburg (Virginia) sand and crushed dolomitic limestone were used for aggregates. Their characteristic properties are outlined in Table 2. The combined aggregate contained 2 to 4 percent more sand by weight than that needed for maximum specific weight.

TABLE 2—MATERIALS

	Sand	Stone, in.		Combined aggregate	
		¼	⅜	44 percent sand by wt., ¾ in. panel	42 percent sand by wt., 1⅛ and 1½ in. panel
Spec. wt.—lb per cu ft	97	89	96	109	116
Spec. gravity	2.63	2.70	2.70	—	—
Percent absorp. by wt.	1.3	0.70	0.64	—	—
Fineness modulus	2.64	5.51	6.24	4.25	4.73

North American portland cement was used for the ordinary and Lehigh portland for the high early strength. Properties, as reported by the manufacturers are listed in Table 3.

TABLE 3—CEMENT

	Specific area	Soundness (Expansion)	Set, hr		Strength, psi	
			Initial	Final	3-day	7-day
Ordinary	1742	O.K.	3:10	5:35	—	365
High early	2880	+0.075	2:10	4:00	451	—

TIME-ELASTIC MODULUS VARIATION

The variation of the modulus of elasticity of the panels with time was measured with a sonic analyzer by determining the fundamental

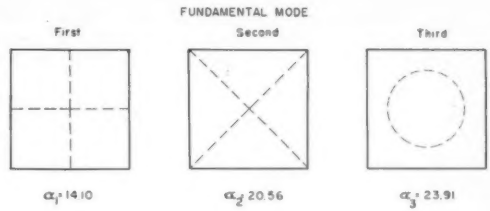


Fig. 3—Theoretical nodal point loci for flat free plates

$$P = \frac{\alpha}{a^2}\sqrt{\frac{gD}{\Upsilon h}}$$

α = a constant
a = the length of a side
g = acceleration of gravity
Υ = specific weight
h = plate thickness
P = frequency

$$D = \frac{Eh^3}{12\,(1-\nu^2)}$$

frequency while the panel was supported on a rubber mat. Readings were taken each time corrosion resistance measurements were made.

Reduction of the frequency to modulus of elasticity is still incomplete. Flat plates do not vibrate in simple sine waves with nodal axes perpendicular to the longitudinal axis as do narrow beams. Instead their nodes depend on the fundamental mode. Theoretical nodal point loci are shown in Fig. 3 for square slabs* and actual measured nodal point loci for the concrete panels in Fig. 4. In any event, the frequency is proportional directly to $\sqrt{E}$, and comparisons can be made. This work is still in progress.

RESULTS OF CORROSION TESTS

Some of the mixes were very lean and dry. They yielded concretes much too harsh for every day practice, but did accelerate this research. They enabled the corrosion to proceed at a rapid rate and established the reliability of the electrical resistance research technique.

The mere plotting of resistance versus time was not satisfactory for it established neither a law, nor a specification by which concretes could be compared. The modification described below does, in the authors opinion, provide such a specification, and also accelerates the test considerably by providing comparative protective ratios for the several concretes at early ages.

HALF-LIFE OF CONCRETE

It is proposed to specify the protection afforded reinforcing steel by any concrete mix and with any specified depth of cover by indicating the

Fig. 4—Actual nodal point loci for concrete slabs

*Timoshenko, *Vibration Problems in Engineering*, D. Van Nostrand Co., p. 423-6.

time required for the reinforcing to corrode so that only half of its original cross-sectional area is left. This time will be referred to as the *half-life* for the concrete. The *half-life* was chosen merely for convenience; any other ratio like *one-tenth-life* could also be used. The authors are indebted to V. G. Szebehely, Applied Mechanics Dept., V.P.I., for suggesting this method of *specifying* a *definite protective life*.

It is possible to determine this half-life accurately from the change in electrical resistance of the steel ribbons embedded in the test panels. To do so, however, it is necessary and convenient to express the change in resistance as a dimensionless parameter.

Since corrosion is proportional to the contact area S, it is also proportional to the perimeter Σo for reinforcing of constant cross-sectional area A and length L. Since Σo decreases as corrosion progresses, the rate of corrosion dS/dt is negative, but its exact relationship to S, and to the geometrical shape factor b of the cross-sectional area is unknown.

Assuming that the electrolytic theory of corrosion holds, $dS/dt \propto S$ is a valid relationship only under certain conditions. These are: (1) that the pitted (corroded) area is distributed somewhat uniformly over area S so that the statistical average may be considered; (2) that the anode and cathode potentials and anode current density remain constant. If (2) holds, the anodic area remains constant, or speaking statistically, proportional to S. Bars and ribbons taken from broken panels which had undergone exposure showed rather uniform pitting. Furthermore, plotted data of corrosion versus time follows the mathematical law derived below, apparently justifying the assumptions made above. The mathematical development of the relationship follows.

Let S = surface area subject to corrosion
A = cross-sectional area of the bar, assumed uniform along its length
a and c = constants
L = length of the test bar
Σo = perimeter of the bar
b = a shape factor = $\Sigma o / \sqrt{A}$
β = a function of several variables including the shape factor, but independent of time
R = electrical resistance (ohms) of the reinforcing at time t
R_o = electrical resistance (ohms) of the reinforcing when the panel is first cast
ρ = R/R_o
t = time
t_o = time after casting when corrosion starts
T = half-life time = time after beginning of the test when $\rho = 2$

Assume $\dfrac{dS}{dt} = -\beta S$. Dividing by L gives

$$d\Sigma o = -\beta\, \Sigma o \quad \ldots\ldots (1)$$

But $$\Sigma o = b\sqrt{A} \quad \ldots\ldots (2)$$

and $$R = cL/A \quad \ldots\ldots (3)$$

Combining Eq. (1), (2) and (3), we get

$$\frac{dR}{dt} = 2\,\beta\,R \quad \ldots\ldots (4)$$

To utilize the resistance in dimensionless form assume

$$\rho = R/R_o \quad \ldots\ldots (5)$$

Substituting in Eq. (4), we find

$$\frac{d\rho}{dt} = 2\,\beta\,\rho \quad \ldots\ldots (6)$$

The exact variation of b as A decreases is a function of the changing geometric shape. For a circle, b remains constant as A decreases due to uniform reduction in diameter. This is also nearly true for a square, but deviates somewhat for other shapes. In the early stages of corrosion, however, it is practically constant for all shapes. It will be assumed constant here. Note that even though this may not be the case, the test data will still give identical *relative* half-life times for the several concretes. Although β is a function of b, it is independent of time. The correlation of plotted data with the law derived here again justifies this assumption. Integrating Eq. (6),

$$\int_1^{\rho} \frac{d\rho}{\rho} = 2\,\beta \int_{t_o}^{t} dt$$

$$ln\,\rho = 2\,\beta\,(t - t_o)$$

For $\rho = 2$, $t = T$; Then

$$2\,\beta = \frac{ln\,2}{T - t_o} \quad \ldots\ldots (7)$$

Hence $\rho = e^{\frac{t-t_o}{T-t_o}\,ln\,2}$ $\ldots\ldots (8)$

To obtain quantitative data on the half-life time for the several mixes, it is necessary to follow the procedure listed below:

(a) Measure R at corresponding time intervals $R = R(t)$

(b) Calculate $\rho = R/R_o$

(c) Plot $\log \rho$ versus time. Then $\log \rho = f(t)$

(d) Read T from this graph at the intersection of the curve and the horizontal line $\log 2 = 0.30103$ (See Fig. 5) .

The half-life thus determined is a specified time only for the particular cross-sectional shape of bar used during the test. The half-life time for any other shape of test bar would be different, and would increase as the shape approaches a circle. If the exact relationship between the function β and shape factor b were known, the half-life for any shape could be determined from the ribbon type tests used. This relationship is not yet known, but its determination should be included in an extension of this program. The relationship will be assumed linear merely for the sake of illustrating an example.

Consider for instance, the 1.5 in. panel 5501. Fig. 6 indicates that $t_o = 16.5$ week, and $T = 37.8$ weeks. Since a ribbon 0.008 x 0.250 in. was used as the reinforcing

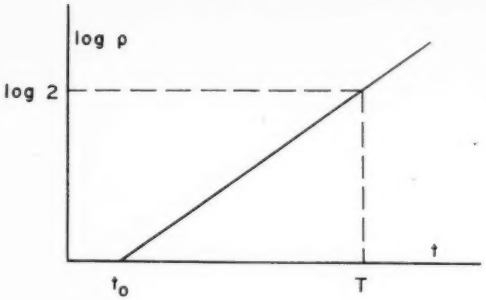


Fig. 5—Relationship of resistance factor to time

$$b = \frac{\Sigma o}{\sqrt{A}} = \frac{2(0.008 + 0.250)}{(0.008 \times 0.250)^{0.5}} = \frac{0.516}{0.0447} = 11.5$$

If T' is desired for a circular area,

$$b' = \frac{\Sigma o}{\sqrt{A}} = \frac{2\pi r}{\sqrt{\pi r^2}} = 2\sqrt{\pi} = 3.54$$

Then, from Eq. 7, taking $\beta = ab$

$$2a = \frac{ln\, 2}{b(T - t_o)} = \frac{ln\, 2}{b'(T' - t'_o)}$$

If $t_o = t'_o$

$$T' = t'_o + \frac{b}{b'}(T - t_o) = 16.5 + \frac{11.5}{3.54}(37.8 - 16.5) = 85.7 \text{ weeks.}$$

It should be evident, from this example, that the thin ribbon serves not only to accelerate the test, so that quantitative data can be determined quickly, but also as a practical resistance element on which sensitive resistance measurements can be made. If $\beta = \beta(b)$ were known, an absolute half-life based on any shape of bar, rather than the present relative half-life based on the 0.008 x 0.250 in. ribbon, could be determined and even specified. Concretes can, however, be just as easily tested, compared and specified with the ribbons.

Data for the various mixes made of ordinary cement showing corrosive action so far are plotted in Fig. 6 and 7. Table 4 lists the half-life times for these panels as determined by the thin ribbons.

Other mixes have so far shown so little corrosive activity that accurate prediction of their half-life is not yet possible, for the slope of the curves is much too flat to hazard its extrapolation. Data for high early cement is not yet available.

EXAMINATION OF CORRODED REINFORCING

So far two panels have developed infinite resistance indicating complete corrosion of the ribbon. These panels and their companion panels were broken open so as to examine both the ribbon and bar. Fig. 8 to 10

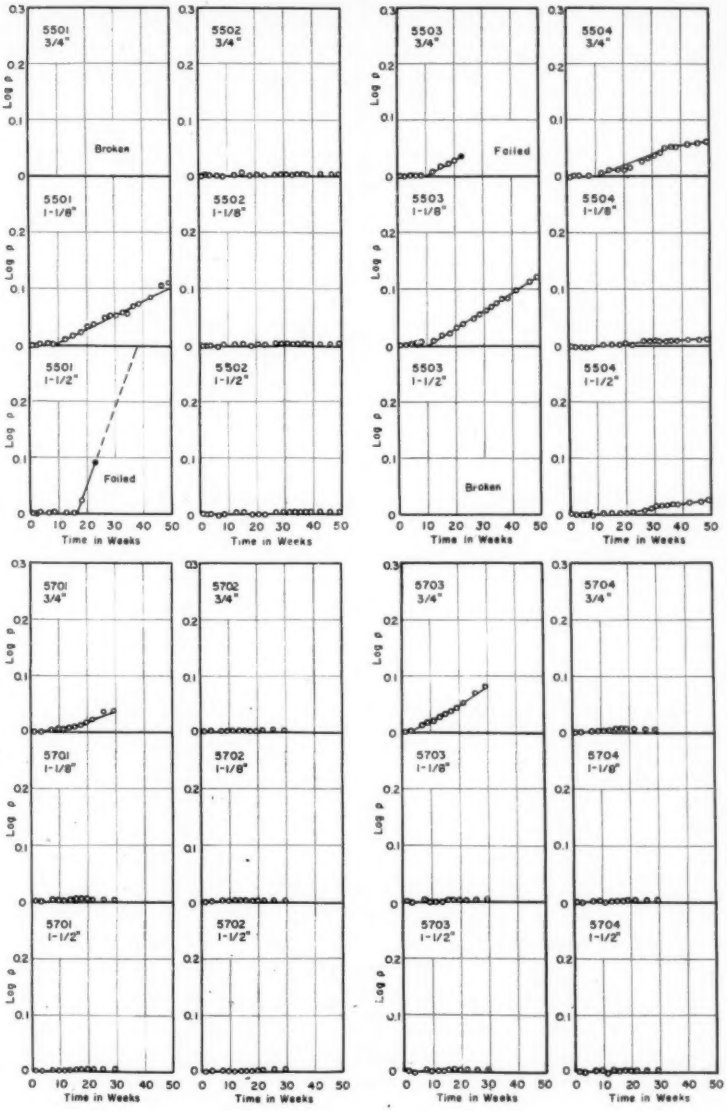


Fig. 6—Corrosive action on 5-sack mixes

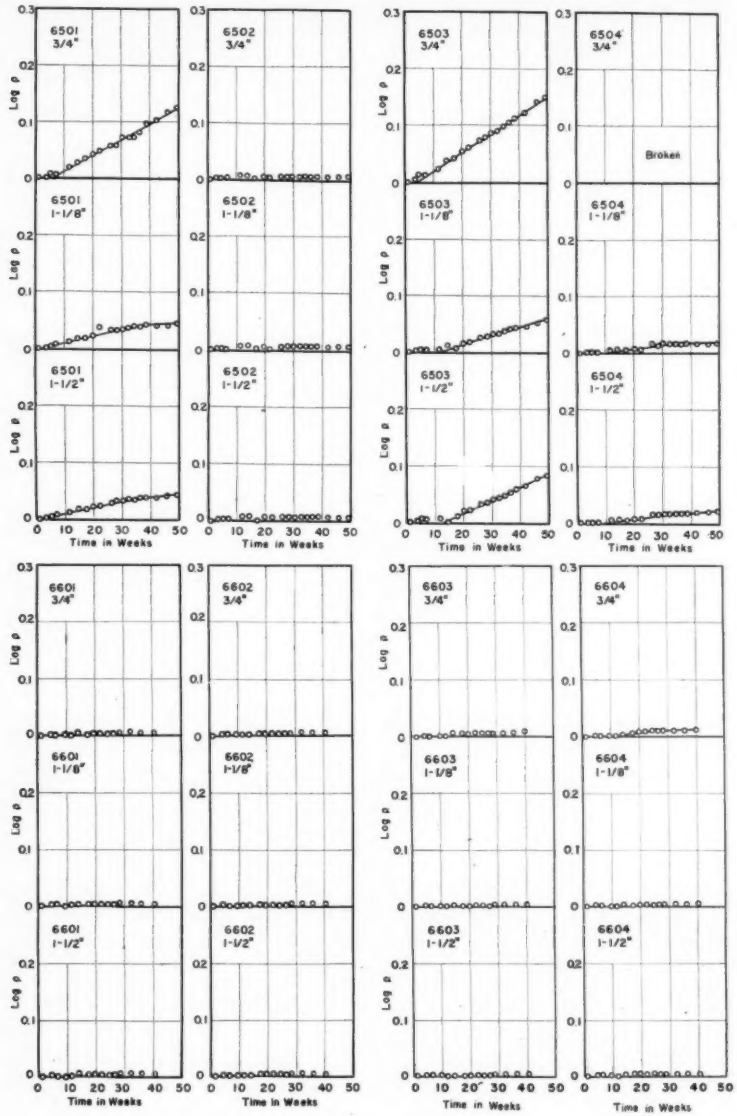


Fig. 7—Corrosive action on 6-sack mixes

TABLE 4—HALF-LIFE IN WEEKS*

Mix	Thickness	Exposure†			
		1	2	3	4
550	0.75	Broken	∞	127‡	155
	1.12	135	∞	106	720
	1.50	37‡	∞	Broken	268
570	0.75	255	∞	116	600
	1.12	∞	∞	∞	∞
	1.50	∞	∞	∞	∞
650	0.75	113	∞	103	Broken
	1.12	235	∞	220	480
	1.50	290	∞	197	480
660	0.75	∞	∞	∞	440
	1.12	∞	∞	∞	∞
	1.50	∞	∞	∞	∞

*Based on a ribbon 0.008 x 0.250 in.
†See p. 515 for exposure conditions.
‡Ribbons completely corroded and panels broken open to examine reinforcing.

show the condition of these. Note the rather uniform distribution of corrosion areas on the ribbon and on the companion bar, and the approximately equal total surface area affected.

Ribbons 6 in. long taken from these panels had a resistance of 0.03200 ohms for panel 5503—¾ in. and 0.01825 ohms for panel 5501—1.5 in., whereas for new ribbons it is 0.01700 ohms.

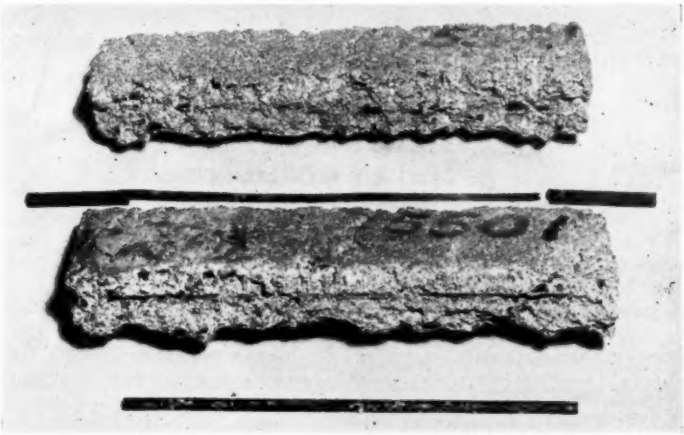


Fig. 8—Panel 5501—1.5 in., showing condition of reinforcing

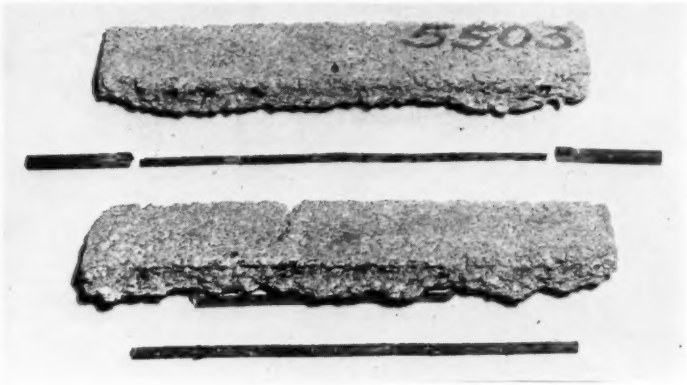


Fig. 9—Panel 5503—0.75 in., showing condition of reinforcing

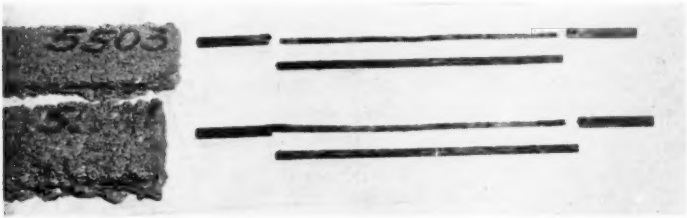


Fig. 10—Comparison of reinforcing in panels 5501 (1.5 in., bottom) and 5503 (0.75 in., top)

RESULTS

Based on the data accumulated so far, the following results seem justifiable:

1. Rich concretes afford better protection than lean concretes.
2. Plastic concretes afford better protection than those placed in a dry condition.
3. Plastic concretes apparently afford complete protection to reinforcing, even when the depth of cover is as small as ¼ in.
4. Exposure definitely affects corrosion. Atmosphere of 50 percent relative humidity or less apparently reduces corrosive action to zero.
5. Depth of cover has some effect, the rate of corrosion decreasing as this cover increases up to $\frac{7}{16}$ in.; thereafter little change is noticeable.
6. The minimum safe cement content in concretes using ¼ or ⅜ in. maximum size aggregate seems to be six sacks per cu yd, provided a plastic concrete is placed.
7. It is possible to predict the half-life of concrete, especially for lean mixes, at a rather early age. In fact, the extrapolation of any sudden break in the ρ-time curve serves to accelerate the accumulation of the necessary data.

8. The half-life time provides a very definite specification of protection which may be incorporated in concrete specifications ultimately when complete data are available.

9. The half-life is a function of the shape of the reinforcing. Thin ribbons may be used to accelerate the test.

10. The electrical resistance ribbon technique apparently yields consistent results.

11. Alternate wetting and drying, even at constant temperature, offers the most severe exposure.

12. The correlation of test data with the theoretical exponential curve as derived seems to justify the assumption between the rate of corrosion and surface area affected.

RECOMMENDATIONS

One objection to the technique described herein is that it takes time. Although it is faster than any other method known to the authors and apparently works, it fails on good plastic mixes. Perhaps these have a half-life approaching infinity, or at least one greater than the service life considering the obsolescence of structures, but that is improbable. The authors considered accelerating the exposure by steam baths, but discarded this because such treatment changes the characteristics of the concrete being measured. Plans are now under way to accelerate the testing by subjecting the panels to fog under 100 psi pressure, followed by an equal time under a vacuum, all at 70 F.

Another objection is that the method does not indicate what influence cracking of concrete may have on the half-life. This phase of the program was omitted purposely so as to eliminate this variable from these initial tests. Future plans call for its inclusion in the test program.

Lastly, the exact functional relationship between the shape factor (b) and the rate of corrosion, is still not known exactly. This too should be included in a future program.

ACKNOWLEDGMENTS

The authors are grateful to the Bureau of Yards and Docks, U. S. Navy, for providing the financial assistance for this project; to A. Amirikian for his valuable advice and interest, to the Virginia Polytechnic Institute Research Foundation for its material aid and to V. G. Szebehely for his suggestion.

Title No. 46-34

Many questions have been raised about hexagonal bins. Here is one method for the

Design of Hexagonal Bins*

By PAUL ROGERS†

SYNOPSIS

The analysis of hexagonal bins is presented with a practical example, showing the details of computations for lateral pressures, inward pull, hanging loads, diaphragm action and the design of the walls, beams and columns.

Hexagonal bins are ideally suited for grain elevators and similar storage facilities. Individual bins or silos may be built economically with a circular cross section, in which case the side pressures are resisted solely by tension.

For larger storage facilities, however, where many bins are grouped together, there are several disadvantages. Either the circular bins are separate, thus wasting valuable space, or they are connected by circular segments. These in turn provide unsatisfactory storage space and produce dangerous unbalanced compression and moments when the adjacent circular bins are empty and the connecting space is full.

Overcoming these difficulties by using continuous square sections has not always proved satisfactory because of the large bending moments in the walls.

Hexagonal bins combine the advantages of both the circular and square bins without their disadvantages. These advantages are: (1) the elimination of curved formwork; (2) low bending moments in the walls due to the short spans; (3) no wasted space; (4) alternately loaded and empty bins have no effect on each other; and (5) favorable "wet perimeter" compared with different cross sections.

The analysis is presented for the design of a bin group, each bin with a hexagonal cross section of 20 ft circumscribed diameter and each side 10 ft long (Fig. 1).

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†Member American Concrete Institute, Structural Engineer, Chicago, Ill.

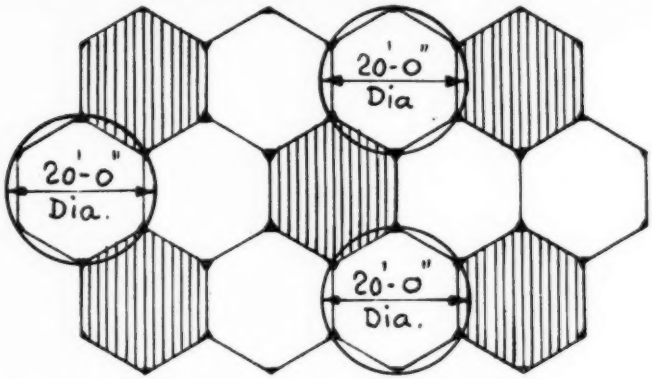


Fig. 1—The general layout of 13 interconnected hexagonal bins, each with a circumscribed diameter of 20 ft. The vertical walls are 80 ft high. At the bottom is a hopper; the vertical walls start 30 ft above ground. The whole bin group is supported on columns, one at each corner.

DESIGN OF A GROUP OF GRAIN BINS

Lateral pressure

Side pressures on the walls are computed by the use of Janssen's formula, although Airy's or other theories can be used at convenience (See bibliography).

Table 1 gives lateral pressures in psf for wheat at 5 ft intervals to a depth of 80 ft for a variety of cross sectional areas. Also see Fig. 2.

The well-known Janssen's formula[1,2,5] gives the value of the lateral pressure p_h at any depth h from the top as follows:

$$p_h = \frac{w R}{\mu'} \left(1 - e^{-\frac{\mu' k}{R} h}\right), \text{ where}$$

w = weight of the stored material (53 lb per cu ft for wheat)
h = depth below top of the point where pressure is calculated
H = total depth of vertical wall, ft
A = area enclosed by a horizontal section, sq ft
U = perimeter of the same section, ft
R = A/U, called the "hydraulic radius"
ϕ = angle of repose, 25° for wheat
ϕ' = angle of friction of material on wall, 24° for wheat
μ' = tan ϕ', 0.444 for wheat

$$k = \frac{1 - \sin \phi}{1 + \sin \phi}, \text{ 0.40* for wheat}$$

e = basis of Napier or natural logarithm, 2.71828
thus

*Approximate value. True value can only be established by experiment. It also varies with the depth and shape of the bin.

$$p_h = \frac{53 \times R}{0.444}\left(1 - e^{\frac{0.444 \times 0.40 \times h}{R}}\right)$$

TABLE 1—LATERAL PRESSURES OF WHEAT ON THE WALLS OF HEXAGONAL BINS*

Circumscribed diam., ft	10	12½	15	17½	20	25	30
A, area, sq ft	65	102	146	199	260	406	585
U, perimeter, ft	30	37.5	45	52.5	60	75	90
$R = A/U$	2.17	2.72	3.24	3.79	4.33	5.41	6.50
Pressure p_h, psf							
at 5 ft depth	87	91	94	95	96	98	99
10 ft	145	157	165	171	175	183	185
15 ft	184	204	219	231	239	254	260
20 ft	210	238	260	278	292	314	325
25 ft	227	262	291	316	334	365	382
30 ft	238	279	315	345	368	408	433
35 ft	245	292	333	368	396	445	476
40 ft	250	301	347	386	420	477	515
45 ft	250	308	357	402	438	503	548
50 ft	250	312	365	413	456	525	577
55 ft	250	312	372	422	466	545	603
60 ft	250	312	376	428	476	560	625
65 ft	250	312	376	435	484	573	645
70 ft	250	312	376	438	491	586	662
75 ft	250	312	376	438	496	596	676
80 ft	250	312	376	438	499	603	690

*These values may vary slightly for different wheats.

TABLE 2—HORIZONTAL FORCES IN BINS

Depth intervals	Total horiz. forces H, kips	Total weight of wheat above, W, kips	$60 \times 0.444 \times \Sigma H$ total weight of wheat carried by friction, ΣF	Weight of wheat to be carried below, $W - \Sigma F$
1– 5	H_1 0.24	69	6.4	62.6
5–10	H_2 0.68	138	24.7	113.3
10–15	H_3 1.04	207	52.4	154.6
15–20	H_4 1.33	276	87.9	188.1
20–25	H_5 1.57	344	129.7	214.3
25–30	H_6 1.75	413	176.3	236.7
30–35	H_7 1.91	482	227.1	254.9
35–40	H_8 2.08	551	282.5	268.5
40–45	H_9 2.14	620	339.5	280.5
45–50	H_{10} 2.23	688	398.9	289.1
50–55	H_{11} 2.31	757	460.4	296.6
55–60	H_{12} 2.36	826	523.2	302.8
60–65	H_{13} 2.40	895	587.2	307.8
65–70	H_{14} 2.43	964	651.9	312.1
70–75	H_{15} 2.47	1033	717.7	315.3
75–80	H_{16} 2.49	1102	784.1	317.9
	ΣH 29.43			

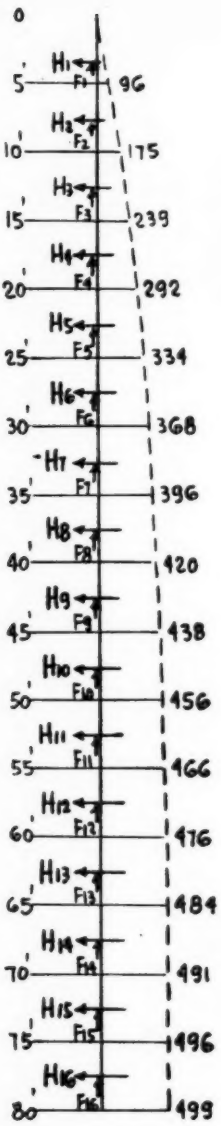


Fig. 2 (left)—Graphic illustration of the lateral pressures of wheat on the side walls of a hexagonal bin with a circumscribed diameter of 20 ft. Values in psf are given for 5 ft intervals as in Table 1. Areas H_1, H_2... are indicated, which correspond to the total lateral forces at each 5 ft interval. The points of action of frictional forces F_1, F_2,...are also indicated. The values of H, F and other useful data are recorded in Table 2.

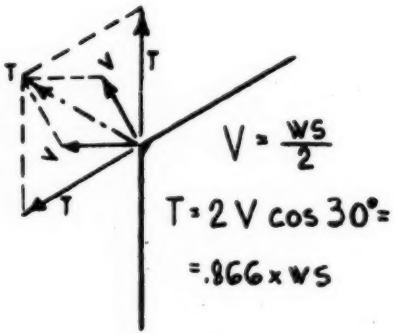


Fig. 3 (above)—Stress development at the corners. The reactions V of the sides acting as slabs are combined and then resolved into two tangential components T which produce tension in the walls.

Wheat load carried by friction by end walls (Table 2), $w_1 = 29.43 \times 0.444 = 13.1$ kip per ft.

Wheat load carried by friction by partition walls $= 2 \times w_1 = 26.2$ kip per lin. ft.

Design of walls at 80 ft depth (see Fig. 3 and 4)

$M_{pos} = 0.5 \times 10^2/20 = 2.50$ ft-kip* $\quad M_{neg} = 0.5 \times 10^2/12 = 4.16$ ft-kip

$T = C = 0.866 \times 0.5 \times 10 = 4.33$ ft-kip $\quad f_c' = 3000$ psi; f_c bearing $= 450$ psi

Maximum vertical load at 80 ft depth per linear ft of wall, (assume 8 in. walls):

w = wheat.....	$2 \times w_1$	= 26.2 kip per lin. ft
wall.....	80×0.10	= 8.0 kip per lin. ft
roof, machinery, etc.		= 2.8 kip per lin. ft
		37.0 kip per lin. ft

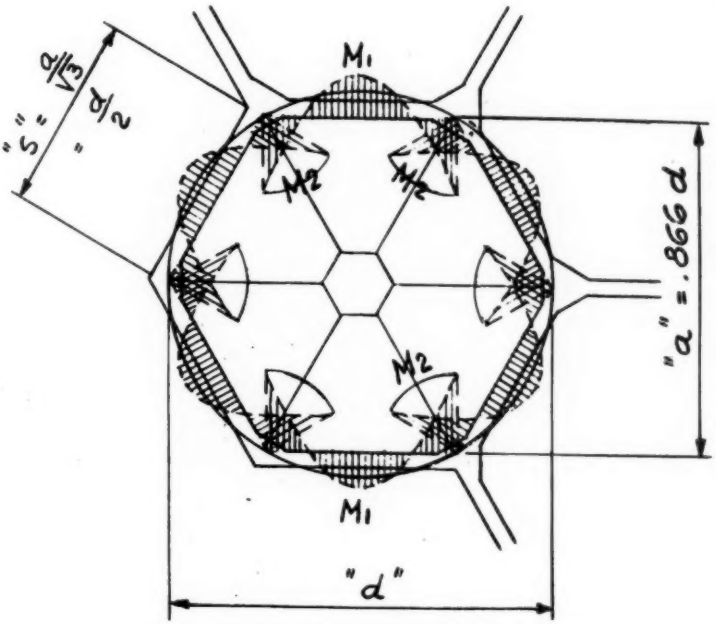


Fig. 4—The general dimensions of each bin and the bending moment diagrams. General formulas for positive and negative bending moments are:

$$M_1 = \frac{P_h s^2}{24} = \frac{P_h a^2}{72} \text{ and } M_2 = \frac{P_h s^2}{12} = \frac{P_h a^2}{36}$$

*The theoretical value is $\frac{0.5 \times 10^2}{24} = 2.08$ ft-kip. It is customary, however, to use slightly higher moments.

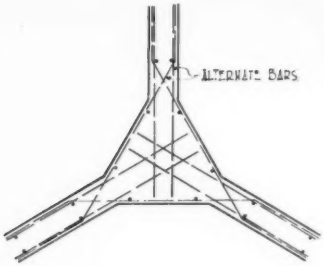


Fig. 5—The recommended haunches at corners and a suggested method for the reinforcing

Concrete thickness required—$37/(12 \times 0.45) = 6.85$ in.; use 8 in. walls, 14 in. at corner haunches.

$$A_{s\ neg} = \frac{4.16}{1.44 \times 12} + \frac{4.33}{2 \times 20} = 0.24 + 0.11 = 0.35 \text{ sq in.}$$

$$A_{s\ pos} = \frac{2.50}{1.44 \times 6} + \frac{4.33}{2 \times 20} = 0.29 + 0.11 = 0.40 \text{ sq in.}$$

Use $\frac{5}{8}$ in. round bars at 9 in. on center, both faces (Fig. 5).

Check for compression in walls between alternately loaded bins—

$$f_{c\ comp} = \frac{4330}{12 \times 8} = 45 \text{ psi, negligible.}$$

Area of steel required for hanging hopper (Fig. 6)—

$$A_s = \frac{7.35}{14} = 0.53 \text{ sq in.; use } \tfrac{3}{4} \text{ in. round bars at 10 in. on center, staggered.}$$

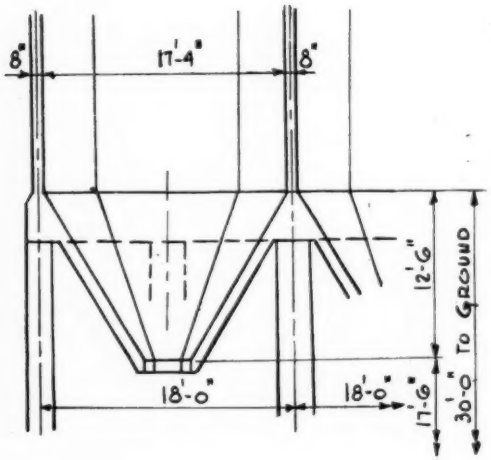


Fig. 6—Load on hopper:

From the bin above	= 318 kip
Wheat in hopper: 1340 x 0.053	= 71 kip
Hopper walls, approx.	= 53 kip
Total	= 442 kip

or $\frac{442}{60}$ = 7.35 kip per lin. ft of perimeter at the intersection of hopper with vertical walls.

Fig. 7—Inward pull:

Vertical hanging load	= 7.35 kip per lin. ft
Tangential component, $\frac{7.35}{\cos 30^\circ}$	= 8.50 kip per lin. ft
Unbalanced horizontal component, or inward pull: 7.35 tan 30°	= 4.25 kip per lin. ft

Low steel stress is recommended here to avoid yield due to pure tension and consequent cracks in concrete. Furthermore, these hanging bars shall be extended for the full depth of the girder carrying part of the vertical walls, in this case 10 ft.

Design of triangularly shaped bottom girder for inward force (Fig. 7)—

The triangular-shaped bottom beams follow the hexagonal pattern in plan view and the moment diagram is similar to that of the walls (Fig. 2), except that the signs are reversed. Similarly, values for tension (Fig. 3) also apply to these beams, except that they act in compression. Thus,

$$M_{pos} = \frac{4.25 \times 10^2}{20} = 21.3 \text{ ft-kip} \qquad M_{neg} = \frac{4.25 \times 10^2}{12} = 35.4 \text{ ft-kip}$$

Compression, $C = 0.866 \times 4.25 \times 10 = 37$ kip Shear, $V = 4.25 \times 5 = 21.15$ kip

According to the *Reinforced Concrete Design Handbook**, Table 4, p. 67, $F = 35.4/236 = 0.15$. Actually a 12 x 16 in. concrete section would suffice; the available concrete section is much larger. Further design is unnecessary, use nominal reinforcing.

Design of bin walls as deep girders (diaphragms)

The bottom part of the vertical walls carry all the loads between columns. These have to be designed as deep beams. The stresses in deep beams do not follow a straight line distribution and the analysis is rather difficult. Fortunately, charts, tables and description of the analysis of deep beams have been published lately in this country.[3]

It has been proved that a height to length ratio of one is the optimum to be used. Consequently, the bottom 10 ft of the vertical walls are designed as deep girders. All coefficients in connection with this design are taken from the above mentioned publication.

*Reported by Committee 317, American Concrete Institute, 18263 W. McNichols Rd., Detroit 19, Mich.

Total load per linear ft—w = 26.2 + 7.35 + 1.45 = 35 kip per lin. ft, or
35,000/12 = 2920 lb per lin. in.
$\beta = H/L = 1$
$\epsilon = C/L = 2/10 = 1/5$
b (width) = 8 in.

Concrete stresses at midspan (Fig. 8):

$p_1 = \frac{2920}{8} \times 0.09 = 33$ psi (comp.)

$p_2 = \frac{2920}{8} \times 0.19 = 69$ psi (comp.)

$p_3 = \frac{2920}{8} \times 1 = 365$ psi (tension)

Concrete stresses at support (Fig. 8):

$p_4 = \frac{2920}{8} \times 0.075 = 27$ psi (tension)

$p_5 = \frac{2920}{8} \times 0.40 = 146$ psi (tension)

$p_6 = \frac{2920}{8} \times 4 = 1460$ psi (comp.)

Although p_6 is slightly above the allowable limit (1350 psi), actually the stresses are lower due to the widening at the bottom.

$v = \frac{35 \times 4}{8 \times 0.875 \times 114} = 186$ psi; $s = 2$ ft; N req'd = $6 \times 2 \times 0.062$ = one $\frac{5}{8}$ in. round bar.

Vertical hopper supporting bars, $\frac{3}{4}$ in. round bars, 10 in. on center, staggered, are satisfactory.

$A_{s\ pos} = \frac{35 \times 10 \times 0.071}{20} = 1.25$ sq in.; use two 1 in. round bars, 8 in. above bottom;

$A_{s\ neg} = \frac{35 \times 10 \times 0162}{20} = 2.85$ sq in.; use three 1 in. square bars, 4 ft above bottom.

Wind forces on columns

Moment of inertia of the column groups:

$I = 2 \times 4 \times 26.3^2 = 5530$

$2 \times 5 \times 21.0^2 = 4420$

$2 \times 5 \times 10.4^2 = 1040$

$2 \times 6 \times 5.3^2 = \frac{340}{11{,}330}$

$S_1 = \frac{11{,}330}{26.3} = 430$

$S_2 = \frac{11{,}330}{21} = 540$

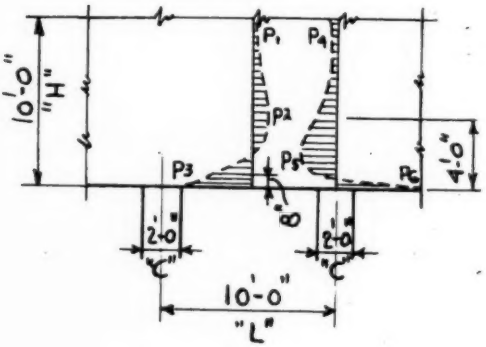
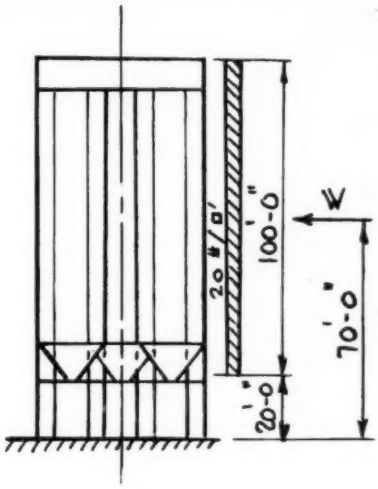


Fig. 8—Diaphragm stresses

Fig. 9—Wind forces



Total wind force (See Fig. 9), $W = 90 \times 100 \times 0.020 = 180$ kip
Overturning moment, $M = 180 \times 70 = 12{,}600$ ft-kip

Maximum vertical wind load on exterior columns, $P'_w = \frac{12{,}600}{430} = 29.3$ kip

Maximum vertical wind load on next row of columns, $P''_w = \frac{12{,}600}{540} = 23.3$ kip

Exterior columns	*All other columns*
$P' = 37 \times 2 \times 5 = 370$ kip	$P'' = 37 \times 3 \times 5 = 555$ kip
$P'_w =$ about 30 kip	$P''_w =$ about 25 kip
Use 24 x 24 in. section, and eight 1 in. round bars.	Use 28 x 28 in. section, and eight 1 in. square bars.

Use 3750 psi (f_c') concrete for columns.
Use tie-beams at 17 ft 6 in. above ground.

Use 12 long, 45° haunches at the intersections of columns and beams to provide adequate bearing surface.

Foundations

The type of foundation (spread footings, continuous footings or pile foundations) is entirely dependent on local conditions and are not included in this paper.

CONCLUSIONS

The foregoing example proves the feasibility and economic value of hexagonal bins, and it is hoped that this type of structure will gain wide acceptance.

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Discussion of a paper by Paul Rogers:

Design of Hexagonal Bins*

By G. L. BLANCHARD and AUTHOR

By G. L. BLANCHARD†

The paper on bin design should prove useful to every designer who has to deal with the problem of deep hexagonal bins. The author is to be commended for the practical point of view which appears to have guided him through the development of the design method.

However, the writer feels that the author should have mentioned the various methods and formulas for evaluating the intensity and distribution of the lateral pressures on walls. In the bibliography he listed two recent studies on silo design[1,2] but made no comparisons with the method used in the paper.

The writer has compared the values obtained through the use of the formulas of Caquot and Kerisel and Reimbert with those of Janssen in Fig. A. To enable the reader to make a practical comparison, the same assumptions that were used by Rogers as to the characteristics of the material have been applied to a hexagonal structure circumscribed in a 20 ft diameter circle.

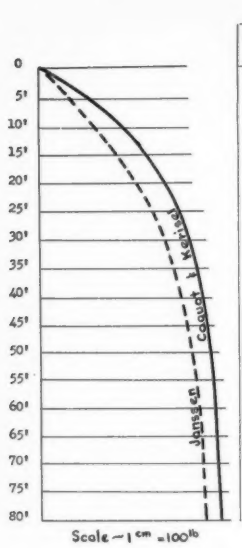
Fig. A shows that the pressures calculated by the two methods of Caquot and Kerisel and Reimbert are much greater than the corresponding pressures in Table 1 obtained through Janssen's formula. It is found that at a depth of 10 ft Caquot's and Reimberts's pressures are 152 percent and 142.5 percent of Janssen's, respectively; at a depth of 20 ft, 134 and 130 percent; at a depth of 40 ft, 117 percent; and at a depth of 80 ft, 110 percent.

The widely divergent results in the upper zones appear theoretically disturbing. Although from a practical standpoint these differences would seldom lead to disastrous consequences, they should not be disregarded.

The conclusions of Reimbert, and the corresponding method and formulas, were evolved after thorough tests and experimental research in France to explain the disorders in existing structures which the accepted methods of design and evaluation of stresses had failed to elucidate. From 1941 to

*ACI JOURNAL, Mar. 1950, *Proc.* V. 46, p. 529. Disc. 46-34 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 4, Dec. 1950, Part 2, *Proceedings* V. 46.

†Chicago, Ill.



Horizontal pressure on wall, psf

Janssen	Caquot & Kerisel	Reimbert
96	151	
175	249	267
239	321	
292	382	390
334	425	
368	451	
396	475	
420	494	491
438	506	
456	516	
466	524	
476	531	
484	536	
491	540	
496	544	
499	547	549

Characteristics of typical cell:
- Radius of circumscribed circle —10 ft
- Depth of cell—80 ft
- Grain:
  - Angle of repose—25°
  - Angle of friction on wall —24°
  - Specific weight of grain—53 lb per cu ft

The typical assumptions used in the numerical computations are those utilized in the original paper in the March JOURNAL. The figures obtained through the use of Janssen's formula are those of Table 1 for a circumscribed diameter of 20 ft.

The pressure values shown in the second and third columns were obtained by using the formulas and methods of Caquot and Kerisel (*Mecanique des Sols*, Paris, 1949) and Reimbert (*Circulaire*, Serie 1, No. 2, Institut Technique du Batiment et des Travaux Publics, Paris, 1943).

Fig. A—Diagram of intensity and distribution of lateral pressure

1943 Maurice Buisson, chief engineer of the civil engineering department of the Bureau Veritas, made further research on the same subject. Though conducted differently, both from the mathematical and experimental standpoints, his investigations corroborated the conclusions of Reimbert. The formulas evolved by Caquot and Kerisel, though they proceed mostly from purely mathematical developments, follow within a small percentage margin the results obtained with Reimbert's formulas.

The designer should also keep in mind the considerations put forward by the various authors, which are summarized below.

The material in storage (grain) is liable to:

1. Increase its specific weight with storage time and in relation to the variations in moisture content. This implies a modification of the internal friction characteristics.
2. Create shear forces during this consolidation (concentration process) along the face of the walls.
3. Change the intensity and distribution of vertical and lateral pressures during the unloading process through bottom openings, depending upon the creation of a vertical funnel of flowing grain starting from the top. This phenomena would vary with the degree of dryness of the grain.
4. Create arching action temporarily under certain moisture conditions, principally at the junction of the vertical walls with the hopper. The rupture of this arch can originate dangerous impact actions.
5. Have its angle of friction along the walls decrease considerably as the repetition of unloading operations polishes the concrete surface.

During loading, the vertical and lateral pressures vary widely in intensity and distribution whether the loading is done slowly or rapidly. Independent of the forces exerted by the stored material, the effect of wind should be investigated thoroughly as outside pressures can be created which may add up with the grain pressures. Due to the generally monolithic character of the structure, the repetition of unequal contraction and expansion strains, mainly on the face of the silo exposed to sun rays, must also be evaluated as to its effect on the acceptable fatigue limit of the concrete.

This enumeration of the numerous factors the designer is bound to contend with may appear somewhat discouraging. But consider the fact that long before the studies of Caquot, Reimbert and Buisson were published, many engineers were aware of the shortcomings of Janssen's formula and supplemented it by correction factors, based mostly on tests.[3,4]

Therefore, if the engineer is well aware of all the factors involved, he has at hand all the implements to solve the problem successfully. The hexagonal type bin is far less vulnerable to most of the ills listed above as there is practically no possibility of dangerously oversimplifying the design of a hexagonal cell alone or in a group.

The writer quotes the optimistic view expressed by Henri Lossier, consulting engineer and chairman of the symposium where Caquot, Buisson and others discussed the conclusions of Reimbert, "Several engineers among the audience may have the impression . . . that the determination of the pressures exerted by pulverized or granular material on the walls of silos is still very uncertain and that dangerous loopholes exist. Let them be reassured. The realizations of the builders, which to an overwhelming majority gave satisfaction, prove that common sense, allied to experience, is perfectly able to cope with the main shortcomings of our theories."

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AUTHOR'S CLOSURE

There can be little disagreement as to whether the analyses of Caquot, Reimbert and Buisson are more accurate than Janssen's formula in its original form. There are, however, several important reasons why the author did not base his computations on these newer theories and had to be content to mention them in the bibliography only.

1. The purpose of the paper was to prove the practicability of hexagonal bins as against the conventional circular cross sections. For comparison, all computations had to be based on similar assumptions. Consequently, Janssen's formula, which is almost exclusively used in this country, was applied.

2. Even in a condensed form, the presentation of the French theories would have required numerous additional pages.

3. The nomenclature and the mathematical presentation by the French authors is materially different from the systems applied in this country, and considerable work would be required to translate their work.

4. In the case of hexagonal bins, the pressures by Janssen's formula or by the French formulas differ but little. It is true that the French values are about ten percent higher than those of Janssen's at the bottom of the bins, but this is due mostly to an often neglected mathematical incongruity.

The "hydraulic radius" is described as the ratio of the area to perimeter. Thus, if r is the radius of a circle, then the hydraulic radius R is,

$$R \text{ circle} = \frac{3.14 \times r^2}{3.14 \times 2r} = \frac{r}{2}$$

If r is also the inscribed radius of a hexagon, then

$$R \text{ hexagon} = \frac{3.464 \times r^2}{6 \times 1.155 \times r} = \frac{r}{2}$$

This implies that in spite of the larger area of the hexagon, the pressures are equal to those of a circle. This, of course, can not be true. The additional volume of stored material is bound to exert both vertical and horizontal pressures which should be added to the pressures of the circular section.

The ratio of the areas of a hexagon and its inscribed circle is

$$\frac{\text{Arae of hexagon}}{\text{Area of inscr. circle}} = \frac{3.464 \times r^2}{3.1416 \times r^2} = 1.10$$

By increasing the pressures of Janssen's formula by ten percent, values practically identical to those of the French authors are obtained. It is to be noted that Reimbert suggests a coefficient of 1.10 for hexagonal bins.

The forces exerted by the grain are definitely altered by moisture content, consolidation, unloading, etc., and Blanchard gave valuable service by enumerating them. It is probably not necessary to include them in the calculations, however, inasmuch the working stresses of both concrete and steel include a favorable safety factor.

The author agrees wholeheartedly with Blanchard that means ought to be provided for the dissemination of newer analytical methods. There are numerous complex structures which require advanced analyses and some of these are only available to engineers acquainted with foreign languages.

Highlights of precast concrete construction as practiced abroad.

Precast Concrete in Britain*

By P. G. BOWIE† and A. R. COLLINS‡

SYNOPSIS

Precast frames for airplane hangars, garages and farm buildings are discussed. Several systems using precast units in housing construction are described and illustrated. Prestressed precast railroad ties and transmission-line poles are mentioned briefly.

The British precast concrete industry, which has been built up over many years, was given fresh impetus by the many new demands made on it during the war. Government contracts for army huts, air raid shelter linings and other war-time structures, combined with the shortage of traditional materials, opened up new fields for precast concrete. It has not only been possible to save structural steel and timber but to effect economies in cost.

The greatest advances in recent years have been made in the use of structural precast concrete framing for buildings, finishing slabs for wall surfacing and house construction.

STRUCTURAL PRECAST CONCRETE

Single-story buildings

Precast concrete structural framing has been used for buildings for a considerable time but during the past few years it has made great progress. In the early days of the war it was used for army huts (about 38,000 were constructed) on a design ultimately standardized with simple frames of 18½ ft or 24 ft span spaced 6 ft apart, hinged at the ridge and lap-jointed at the eaves. This form of construction with wider spacings and spans is now in general use for a large variety of buildings including hangars, garages, factories, farm buildings and

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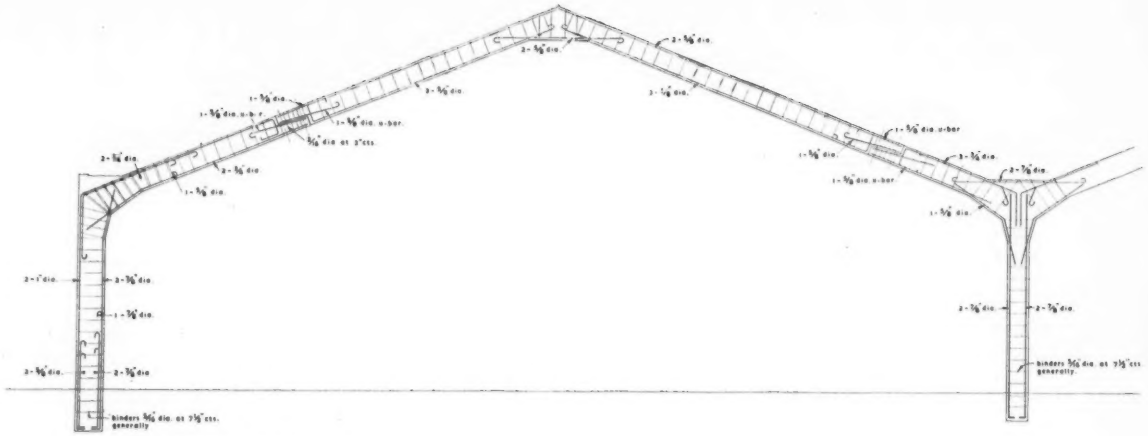
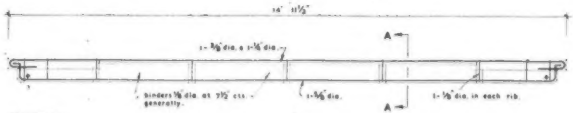
†Member American Concrete Institute, Chief Structural Engineer, Cement and Concrete Assn., London, England.

‡Member American Concrete Institute, Technical Director, Cement and Concrete Assn., London, England.



Typical reinforcement detail of main frame



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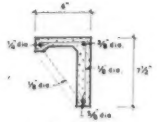


section A-A

Typical reinforcement detail of purlin.

Fig. 1—Details of reinforcement of a precast concrete factory building

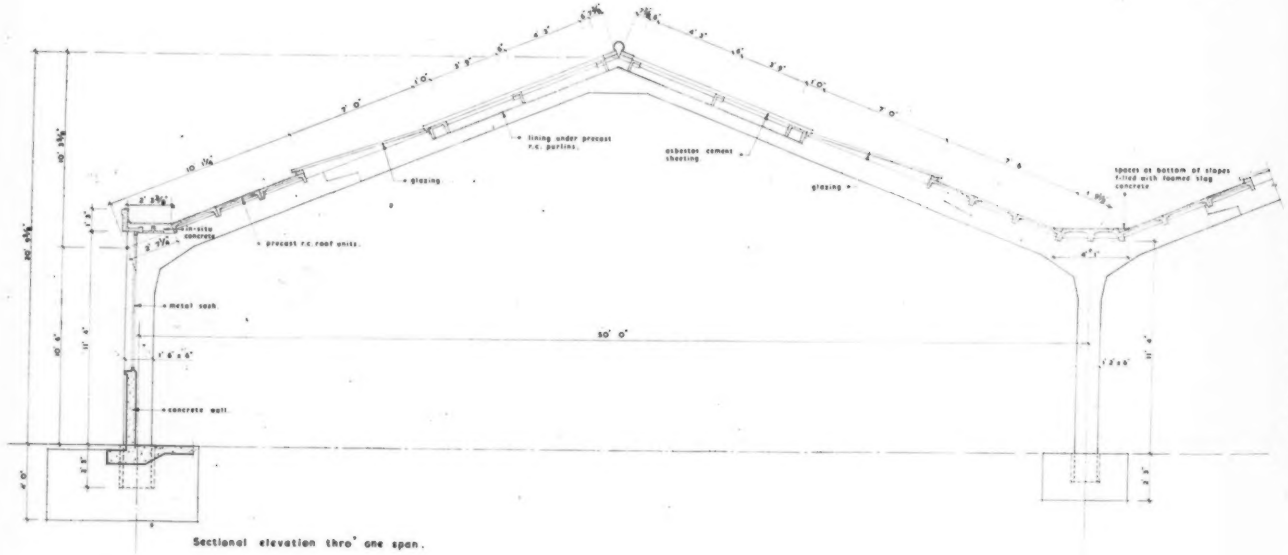


Fig. 2—Construction details of precast concrete factory building

schools. The hinge at the ridge is now often omitted and the joint at the eaves moved up the roof frame to the point of contraflexure, thus making the structure into a type of portal frame.

A typical example of this type of design is shown in Fig. 1. Two main structural units are employed, one in the shape of a large Y which forms the column and lower part of the roof and the other in the shape of an inverted V forming the upper part of the roof. The junction between the two main members, as mentioned above, is placed at the point of contraflexure under dead load conditions and is usually formed by means of a "halved" or scarfed joint bolted together. The building illustrated is 480 ft long by 250 ft wide and consisted of five rows of frames spaced 15 ft apart. The upper part of the roof is covered with corrugated asbestos cement sheeting supported on purlins consisting of precast reinforced concrete angles 7½ in. x 6 in. in section (Fig. 2) while the lower part of the roof is covered with precast concrete slabs, spanning between the roof frames without purlins, the roof between the asbestos cement sheeting and the concrete slabs being glazed in the normal way as shown in Fig. 2. Precast concrete gutter units were used at the eaves and the precast roof slabs form valley gutters between the frames, the upper surface of all the gutters being waterproofed.

All the smaller units were precast at a factory. The first work on the site consisted of casting in situ foundation blocks with sockets to take the columns. The floor slab was then cast in situ; the main frame members were precast on the floor and the ends of the columns placed in the socket in the foundation blocks and later grouted. The roof units were then lifted into place and the joints between them and the columns bolted up after having first been coated with mortar (Fig. 3).

This basic design has been used by the same British firm for the construction of a hangar for the French Ministry of Aviation in which

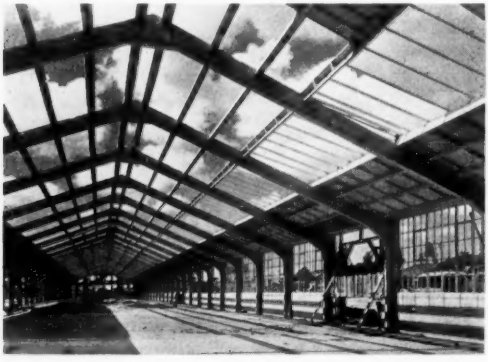


Fig. 3—One span of precast concrete factory building illustrating the characteristic arrangement of jointing. Part of the roof is covered with precast concrete slabs, the remainder is glazed.

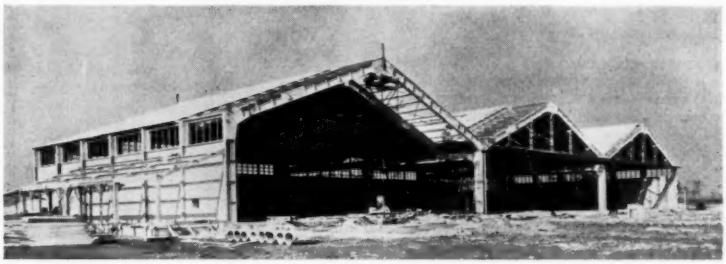


Fig. 4—Precast concrete aircraft hangar built by an English firm for the French Ministry of Aviation. The large units, which span 98 ft 6 in. without intermediate support, were precast on the site.

the spans were 98½ ft (Fig. 4) and similar designs have been used for garages and farm buildings.

Several different types of single-story precast concrete framing are used for school construction, a typical example of which is shown in Fig. 5a. In this instance, the frame consists of precast reinforced concrete columns having a short length of beam cast at right angles at the top to form the beginning of the roof beam. The main roof beams are also precast and are joined to the portion of beam at the top of the column by a short length of concrete cast in situ; the same mold supports the center part of the beam while the concrete is hardening.

The secondary roof beams, the roof slabs, the interior and exterior walls and eaves units are also of precast concrete (Fig. 5b).

Multi-story buildings

Precast concrete multi-story buildings have been used considerably in the past few years but not to the same extent as single-story buildings and the methods used have varied more. In some instances the construction consists essentially of placing a normal type of single-story building frame above a lower story cast in situ. In other instances precast units have been used throughout as, for example, the block of flats shown in Fig. 6 and 7 which was constructed by a process similar to that used in the school shown in Fig. 5 with precast columns and floor beams. The small pieces of beam cast integrally with the columns

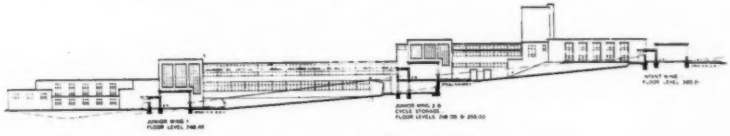


Fig. 5a—A school building in precast concrete

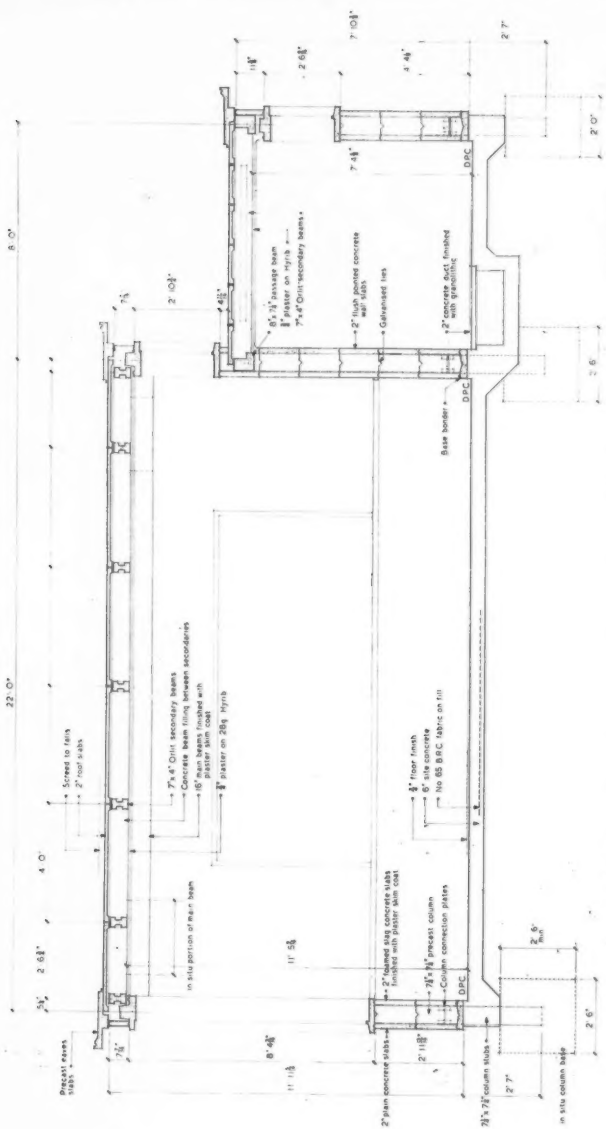


Fig. 5b—Typical section through classroom and corridor of precast concrete school building

Fig. 6 — A three-story block of flats under construction. Note the small sections of beam cast on the tops of the columns.

can be seen in Fig. 6 and the exterior precast surfacing slabs are shown in Fig. 7.

In other methods of construction, precast beams are used with columns cast in situ. Several large blocks of offices have been built in London since the war by this method. The design of these buildings is similar to that of the normal reinforced concrete building, but by precasting all the beams it has been possible to achieve considerably greater speed in construction and a useful saving of forms.

In one or two instances, columns have been formed of precast concrete pipes subsequently filled with in situ concrete and the beams have been partly precast and partly cast in situ (Fig. 8).

Small houses

A great part of the building work undertaken in Britain since the war has been devoted to the construction of small houses. Traditional methods have been unable to meet the demand and many types of temporary and permanent precast concrete houses have been developed. Among the most widely used designs is the "Airey" house, which consists of a frame of closely spaced story-height precast columns on which narrow precast concrete slabs are fixed horizontally with overlapping joints similar to weatherboard timber construction. The columns are

Fig. 7—Three-story block of flats showing walling slabs

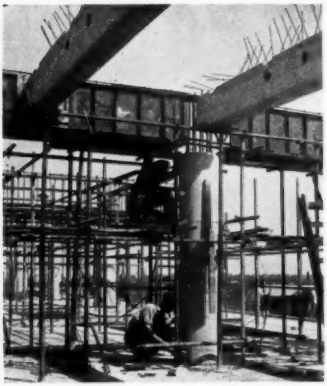


Fig. 8—A corner of a factory building under construction, showing the combined use of in situ and precast beams, and a column built up of precast concrete piping used as permanent formwork for an in situ concrete filling

4½ x 2¼ in. in section and are spaced 18 in. apart. At second floor level, the floor joists are bolted to the top of the first floor columns and the second floor columns joined to them by means of steel pins passing down the center of each column. Roof beams are bolted to the top of the second floor columns in the same way as the second floor beams. The facing slabs are held to the columns by stout copper wires passing around the columns (Fig. 9).

The method used for constructing the three-story block of flats mentioned above and shown in Fig. 6 has also been widely adopted for two-story house construction, with minor modifications.

In a number of types of house construction, precast concrete is used in a more traditional way. In one form, illustrated in Fig. 10, the walls consist of precast load-bearing slabs of story-height, composed of a dense reinforced concrete shell 1¼ in. thick with a backing of light-weight concrete. Floors and roofs with this form of construction are generally of timber in the traditional manner.

Another method uses T-shaped precast concrete wall sections fixed vertically on end, side by side, with the top of the T forming the outer wall and the tail of the T supporting the inner wall. A number of specially shaped precast concrete members are also used with this form of construction which is illustrated in Fig. 11.

Walls

In addition to the methods of wall construction already mentioned, precast concrete units often are used as load-bearing walls and for non-load bearing cladding of framed buildings. These units include precast wall panels of various shapes, cavity blocks or double blocks with the interior and exterior surfaces connected with metal ties and, of course, hollow and solid masonry block.

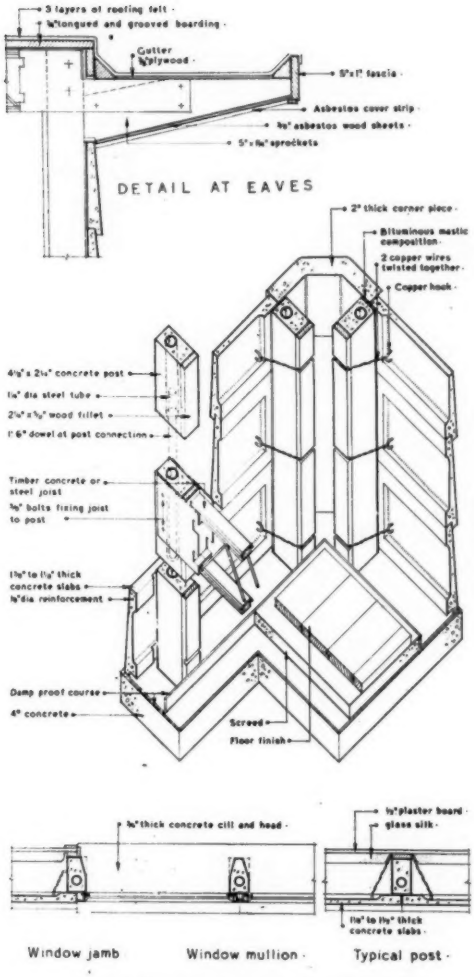


Fig. 9—Construction details of "Airey" house

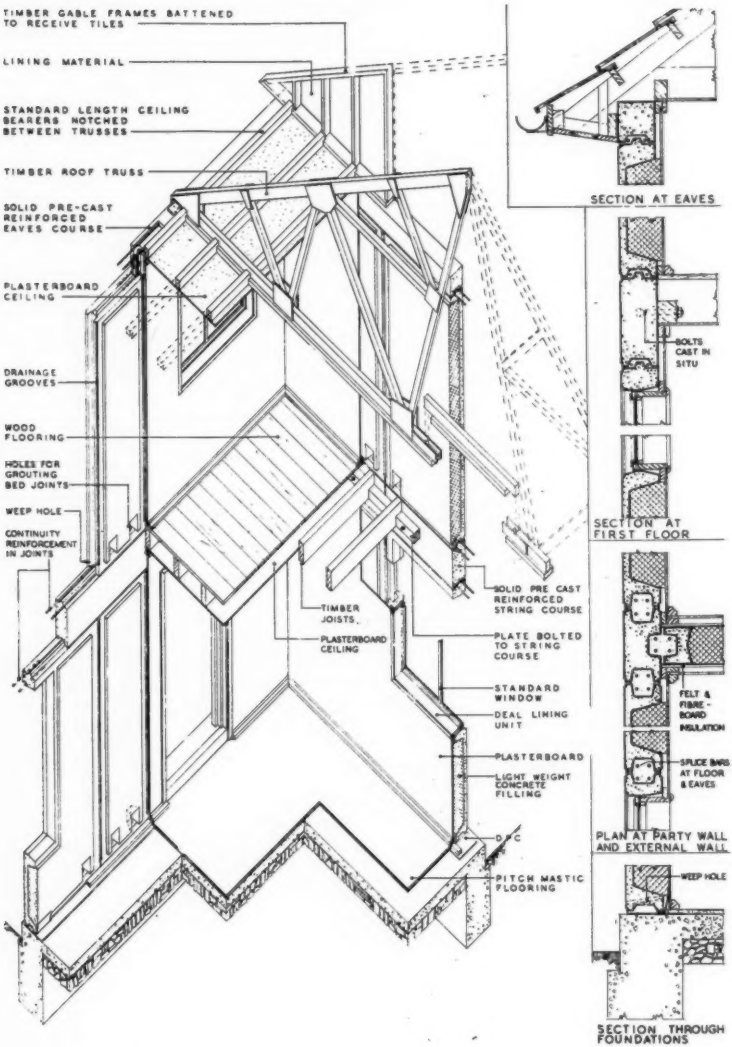


Fig. 10—Precast concrete house construction by the "Wates" system

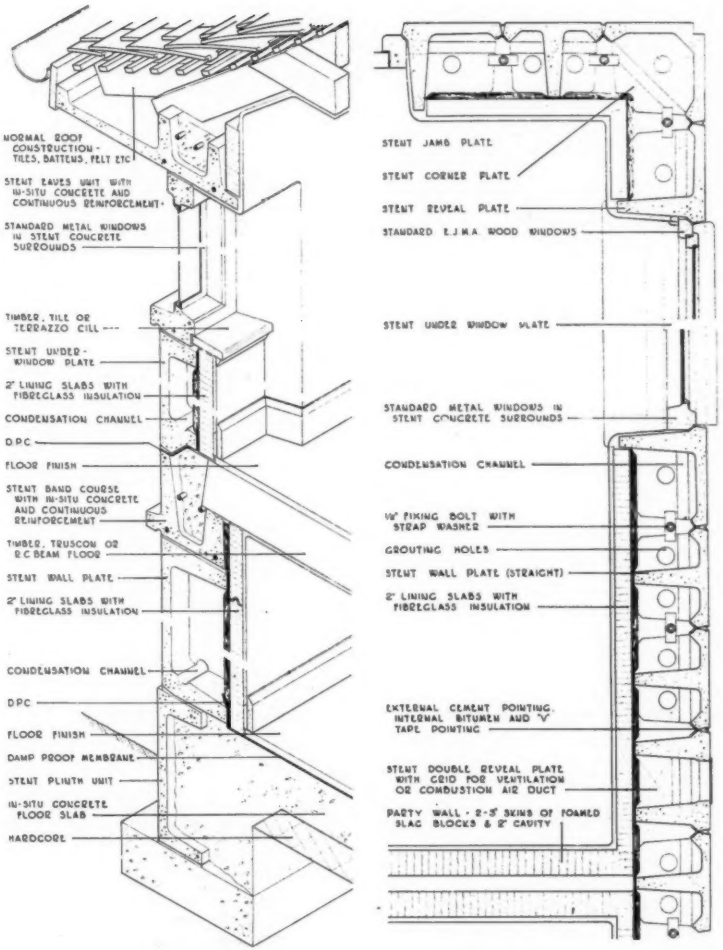

NORMAL ROOF CONSTRUCTION - TILES, BATTENS, FELT ETC
STENT EAVES UNIT WITH IN-SITU CONCRETE AND CONTINUOUS REINFORCEMENT
STANDARD METAL WINDOWS IN STENT CONCRETE SURROUNDS
TIMBER, TILE OR TERRAZZO CILL
STENT UNDER-WINDOW PLATE
2" LINING SLABS WITH FIBREGLASS INSULATION
CONDENSATION CHANNEL
D.P.C
FLOOR FINISH
STENT BAND COURSE WITH IN-SITU CONCRETE AND CONTINUOUS REINFORCEMENT
TIMBER, TRUSCON OR R.C BEAM FLOOR
STENT WALL PLATE
2" LINING SLABS WITH FIBREGLASS INSULATION
CONDENSATION CHANNEL
D.P.C
FLOOR FINISH
DAMP PROOF MEMBRANE
STENT PLINTH UNIT
IN-SITU CONCRETE FLOOR SLAB
HARDCORE
STENT JAMB PLATE
STENT CORNER PLATE
STENT REVEAL PLATE
STANDARD R.I.B.A. WOOD WINDOWS
STENT UNDER WINDOW PLATE
STANDARD METAL WINDOWS IN STENT CONCRETE SURROUNDS
CONDENSATION CHANNEL
½" FIXING BOLT WITH STRAP WASHER
GROUTING HOLES
STENT WALL PLATE (STRAIGHT)
2" LINING SLABS WITH FIBREGLASS INSULATION
EXTERNAL CEMENT POINTING. INTERNAL BITUMEN AND 'V' TAPE POINTING
STENT DOUBLE REVEAL PLATE WITH GRID FOR VENTILATION OR COMBUSTION AIR DUCT
PARTY WALL - 2-3" SKINS OF FOAMED SLAG BLOCKS & 2" CAVITY

Fig. 11—Precast concrete house construction by the "Stent" system

Precast concrete wall panels are frequently used for farm buildings as shown in Fig. 12.

The surface finish of concrete buildings often has been criticized in the past and much attention is now being given to improving the appearance of concrete buildings by the development of special precast surface slabs. The wall panels of some of the prefabricated houses mentioned have special textured surfaces obtained either by exposing the aggregate, by casting against specially designed formwork or by the surface application of various finishes.

Slabs of a similar kind have also been used to surface steel-framed buildings of normal construction as shown in Fig. 13. Precast concrete slabs are also used as permanent formwork for in situ concrete and give an attractive, durable and uniform surface finish. These slabs are of various forms and sizes and generally have a textured finish obtained by exposing the aggregate by scraping, spraying or tooling. Considerable development of this technique is expected in the next ten years.

PRESTRESSED PRECAST CONCRETE

The use of prestressed concrete in Britain began in the early days of the war when it was used in precast beams for military stores and in the form of girders for railway bridge replacements. Railway sleepers (ties) of ordinary reinforced concrete have been used for sidings for many years but were found to be inadequate for main line use. The development of prestressed concrete sleepers overcame this difficulty and now four large factories are producing millions of main line railway sleepers by the "long line" process of prestressing. A design for a sleeper with anchored wires to be made by casting individually is about to be put into large scale production.

Prestressed concrete is now becoming more and more widely used, engineers and government departments being keenly interested in its potentialities. Precast prestressed beams are being used in several current construction jobs, including two important factories; another

Fig. 12—An all-concrete farm in Kent. The buildings are concrete-framed and the wall infilling consists of horizontally-placed precast concrete panels. The roofs are covered with corrugated asbestos cement sheeting.

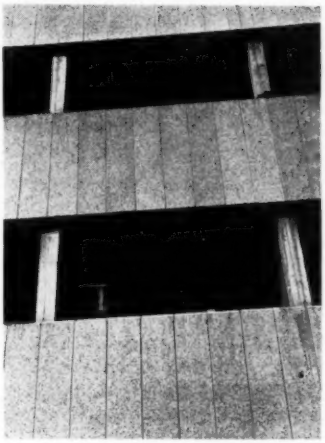


Fig. 13—Facing slabs used on a newly completed block of Council flats in London. The narrow vertical panels, placed edge to edge, form with the continuous strips of glazing an effective pattern of horizontal stripes. This is a steel-framed building.

factory which uses prestressed columns is also under construction. A notable point about this design was the use of columns made up of short precast hollow sections which were assembled on the site and prestressed by means of four Freyssinet cables running vertically from top to bottom. A column of this type is shown in Fig. 14a and b. The column sections were about 5 ft long and were cast in the factory and assembled at the site on a horizontal platform. The four cables were threaded through the holes provided for them and the joints made with a ¾ in. layer of stiff mortar. After 24 hours the cables were stressed by the normal Freyssinet method with a load of about 20 tons in each cable. The cable holes were later grouted to prevent corrosion and slipping.

This method of casting members in sections and assembling them by prestressing has been used in bridges in England but the most notable structures in which it has been employed are the bridges over the Marne designed by Freyssinet. It offers possibilities of much greater development for the precast concrete industry as it allows large structures to be built up of sections small enough for easy transport but requires concrete of high quality that can best be produced in a well-organized precast products factory. The method also has the possibility of producing standardized sections suitable for a variety of uses.

Another interesting building in prestressed concrete, where the floor loadings are particularly heavy, is now being erected in Scotland using columns cast in situ and precast beams prestressed by the Magnel method. All the beams are cast and prestressed on the floor and lifted into place by crane. Additional prestressing cables are then passed

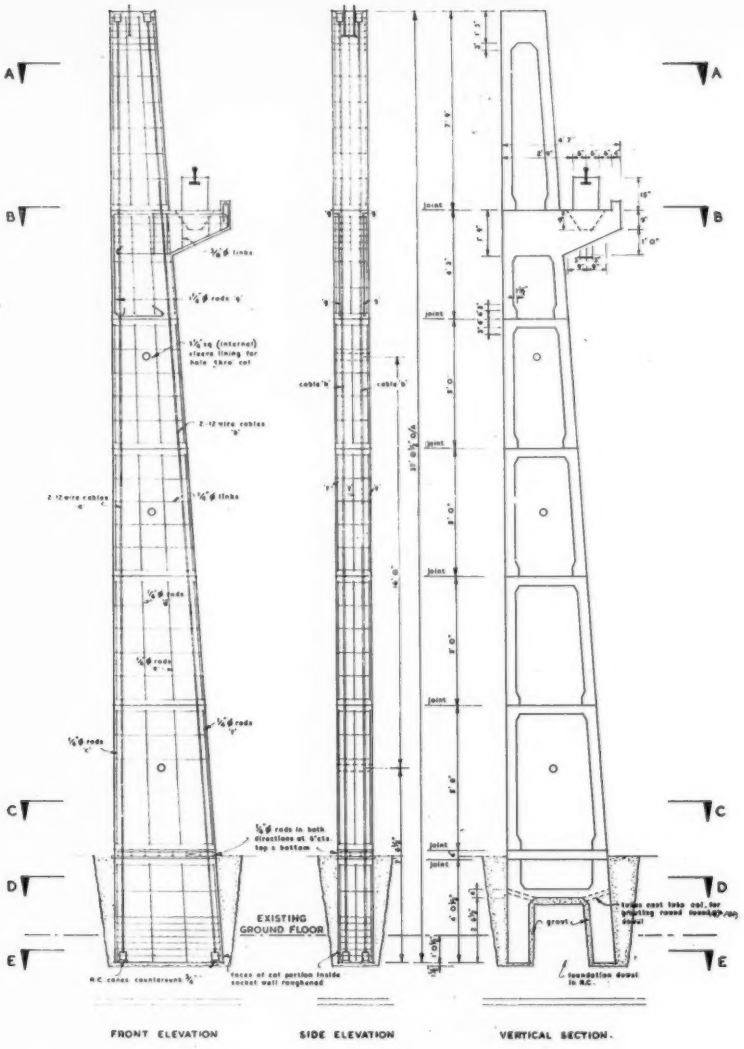


Fig. 14a—Details of a precast prestressed column used in a factory construction

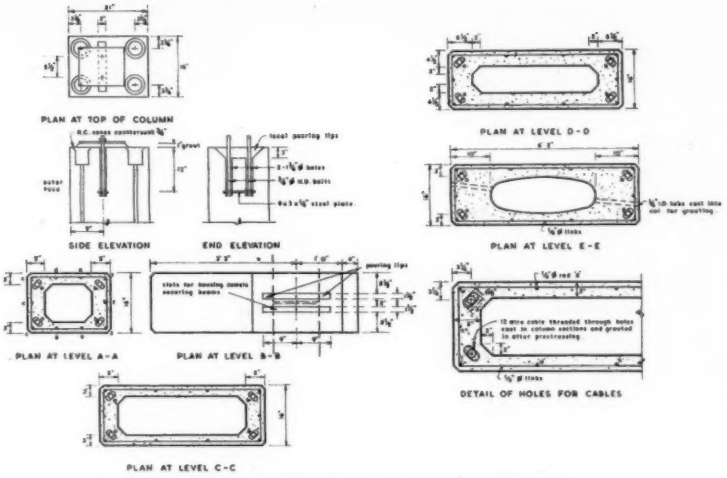


Fig. 14b—Further details of precast prestressed column

right through the beams from one side of the building to the other so that the whole structure is given rigidity and structural continuity. The normal Magnel methods of design and construction are used.

A number of factories have been set up since the war to manufacture precast prestressed units for the building trade, particularly hollow floor beams, as well as such products as transmission poles and lighting standards (Fig. 15).

ROADS

The prestressed concrete runway at Orly airdrome, near Paris, is an example of the use of precast road slabs and a similar project has been under discussion for Britain. But the furnishing of a road—its curbs, sidewalks, fencing and lamp standards, provides a considerable market for precast concrete. Curbs in Britain are usually of a simple form, defined by British Standard Specification. Sidewalks of precast slabs, 3 x 2 ft and about 2 in thick, are customary here, partly because the telephone, water and other services are generally laid below the path, partly because of the ease with which such paving can be replaced or leveled, but principally because they stand up better than any other form of paving to continuous heavy wear.

ROOF COVERINGS

The use of concrete roofing tiles has continued to grow in favor until the output has now equaled that of clay tiles. Mechanized production

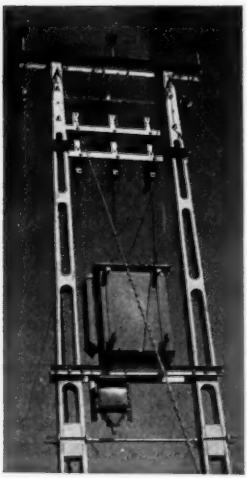


Fig. 15—Prestressed concrete transmission-line pole. This pole which supports a transformer consists of two normal poles bolted together.

and the study of coloring methods, added to a guarantee of replacement if defective, have been largely responsible for the increasing demand.

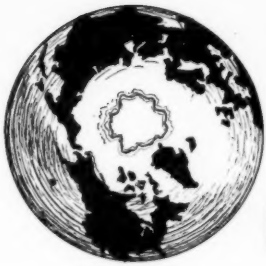
For industrial and farm buildings, asbestos cement sheets, either plain or in a variety of corrugations of different design and size, are generally used, fixed to purlins 4 ft apart. For the flat roofs of schools, where better insulation and a span up to 8 ft between transverse beams is required, a lightweight decking built up of a special double-skin sheeting covered with bituminous felt or some similar material is frequently used.

CONCLUSION

The precast concrete industry in Britain has made great strides in the last ten years, but past progress seems of small importance compared with the possibilities opening up before it today. The application of prestressing to precast products is immeasurably increasing their scope, and the increased use of lightweight concrete provides new opportunities for the industry. Prestressed bridge sections are at one end of a scale that takes in pipes of all sizes and reaches down to such apparently simple objects as concrete fence posts where it is anticipated that prestressing will produce a much stronger and more durable product.

ACKNOWLEDGMENT

We wish to acknowledge the valuable assistance R. W. Parks, Secretary of the British Cast Concrete Federation, has given to us in the preparation of this paper.



Letters from Readers*

BY WAY OF SYNOPSIS

M. W. JACKSON discusses the design of paneled ceilings.

For those who secure furring strips to concrete blocks S. A. MEDER discusses holding power of nails.

GEORGE W. BLANCHARD recommends a series of University of Liège publications.

The proportioning of footing areas is demonstrated by CHU-KIA WANG.

Paneled Ceiling Loaded by Concentrated Loads at the Intersections of the Beams (LR 46-37)†

M. W. JACKSON, School of Civil Engineering, Georgia Institute of Technology, Atlanta, Ga., points out cautions which must be applied to the design solution offered in the December JOURNAL.

The solution offered by L. S. Muller in "Paneled Ceiling Loaded by Concentrated Loads at the Intersections of the Beams" (LR 46-37) is of interest, but it would be better if all of its limitations were completely stated. To attempt to apply the formula derived by the author to reinforced concrete design, it is absolutely essential to understand these limitations.

The solution is based on beams of span L_x and L_y which must be supported on unyielding foundations. The solution is applicable only for buildings with dimensions $L_x \times L_y$ with concrete or masonry walls, and it is not applicable for buildings with bays $L_x \times L_y$. The deflection of girders between columns of bays would significantly affect the distribution factors, α.

Simple supports are required at the ends of the beams, but concrete beams which are part of a building are rarely free at the ends. If beams

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 7, Mar. 1950, *Proceedings*, V. 46. Separate prints are available at 35 cents each. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†See also ACI JOURNAL, Dec. 1949, *Proc.* V. 46, p. 293.

were continuous through girders, the deflections of the beams could no longer be computed for simple supports, but the effect of the moments in the beams at the intersection of the beam and girder must also be considered. Similarly if the beams frame into columns, the end moments will be important.

Another design complication is introduced because it has been assumed that the moment of inertia is constant throughout. Any design should be examined carefully to see if this is justified. Cutting off bars where they are not needed to carry moment will change the characteristics of the beam. If there is an integral floor slab, it is difficult to justify a uniform moment of inertia for a ceiling with a small number of beams. Since the specification limits for flanges of T-sections are not necessarily applicable in calculating deflections, evaluating the varying moment of inertia presents another difficulty. If there is any negative moment at the support (or elsewhere on the span), it will be almost impossible to meet the condition of equal moments of inertia throughout, with part acting as a T-section and part as a rectangular section.

The assumption pointed out by the author that the torsional resistance of the beams has been neglected is satisfactory for beams of the usual proportions, but in some cases it may be necessary to consider that the angle of rotation of one beam at an intersection must be equal to the slope of the intersecting beam at that point. This would introduce as many additional unknown quantities into the solution as there are distribution factors, α.

Holding Power of Nails in Concrete Blocks (LR 46-52)

S. A. MEDER, Wheeling Steel Corp., Wheeling, W. Va., gives data on driving and holding ability of nails in several types of concrete blocks.

Users of concrete blocks who fasten wood furring strips to them will find of interest the results of Pittsburgh Testing Laboratory tests of the nailability of several types of concrete blocks using common and cut nails and the holding power of the nails after driving.

Table 1 gives the characteristics of the blocks used in the tests. While the age at which tests of compressive strength were made varied widely a limited comparison of this property is possible.

Duplicate tests were made with seven penny cut nails and with common nails of the same nominal size. The nails were carefully driven through 1 x 2-in. furring strips into the building block which was rested in a sand bed to prevent undue cracking caused by the impact of hammering

TABLE 1—PROPERTIES OF CONCRETE BLOCKS

Type	Size, in.	Wt. of concrete per cu ft, lb (dry weight)	Approximate age at time of test	Compressive strength, psi (gross area)
Sand and gravel	8 x 8 x 16	138.7	14 days	1700
Celocrete	8 x 8 x 8	94.2	18 months	1598
Slaglite	8 x 8 x 16	109.3	28 days	1189
Pumice	4 x 8 x 16	63.6	13 months	882
Cinder	8 x 8 x 16	92.8	26 days	1055

the nail. For the first five tests, white pine furring strips were used. However, for the sake of comparison with previous test results, yellow pine strips were used for all subsequent tests. The furring strips were cut with beveled edges to facilitate gripping of the strip when pulling the nails out. A suitable fixture fitted over the beveled edges of the strips and the load was applied vertically through this fixture by means of a Baldwin 200,000 lb capacity hydraulic testing machine. The 5000 lb range of the machine having 5 lb as the smallest division was used for all loading. In the final tests, 10 tests were made using each type of nail and each type of block.

NAILABILITY TESTS

Table 2 shows a comparison between the driveability of common and cut nails into several types of concrete block.

TABLE 2—NAIL DRIVING TESTS

Type of block	Type of nail	
	Cut	Common
Sand and gravel	1 nail fractured half way into block. 6 nails bent after driving half way.	7 nails bent after striking block.
Slaglite	Drove easily	5 nails bent shortly after entering block.
Pumice	Drove easily	Bent slightly but all drove in.
Cinder	Drove easily	¼ of nails drove, rest bent.
Celocrete	Drove fairly easily	Less than ¼ drove, rest bent.

NAIL PULL TESTS

Tables 3 and 4 give the maximum loads necessary either to pull out the nails or destroy the furring strip. White pine strips were used for the tests in Table 3 while yellow pine was used for the rest.

TABLE 3—PRELIMINARY TESTS OF CUT NAILS DRIVEN IN CELL WALL

Test No.	Type of block	Maximum load, lb	Remarks
1	Pumice	125	Wood pulled from nail
2	Celocrete	135	Wood pulled from nail
3	Cinder	180	Wood pulled from nail
4	Slaglite	220	Wood pulled from nail
5	Sand and gravel	190	Wood pulled from nail
6*	Sand and gravel	250	Nail and wood pulled from block

*Yellow pine strip was used for this test.

TABLE 4—HOLDING POWER OF NAILS DRIVEN INTO CENTER CELL

Type of block	Maximum load, lb*	
	Cut nails	Common nails
Sand-gravel	245	—†
Slaglite	275	—†
Pumice	195	25
Cinder	230	50
Celocrete	225	85

*Average of ten tests
†No test

In most cases, nailheads pulled through the wood in tests of nails driven into sand-gravel and Slaglite blocks. With pumice blocks both cut and common nails generally pulled out of the block. This tendency was less marked with cut nails driven into cinder blocks where about half the time the nail head pulled through the wood. In the tests on Celocrete blocks all the cut nails pulled through the furring strips while common nails pulled out of the blocks.

Notes on Seven Universite de Liege Pamphlets, and a Paper from the Preliminary Publication of the Third Congress of the International Association of Bridge and Structural Engineering

(LR 46-53)

GEORGE L. BLANCHARD, Consulting Engineer, Chicago, makes interesting comment on Belgium publications.

JOURNAL readers may be interested to know that the Civil Engineering Division, University of Liège (Belgium), has issued pamphlets dealing with subjects of fundamental importance in civil and structural engineering. These papers are published under the signature of Prof. Fernand Campus, world wide authority in reinforced concrete and stability of structures, and member of ACI, and of several of his assistants.

They have the remarkable merit, even for the practicing engineer, of acquainting the reader, in a few pages, with subjects of basic scientific value and with the most recent developments of the new techniques.

For instance, the latest papers deal with laboratories for testing structures (No. 60), inspection, tests and measurements on existing structures (No. 61), the control and regulation of stresses in structures (No. 64), tests on materials (No. 65), tests on models of structures (No. 67), prestressed concrete (No. 68), and the importance of secondary stresses (No. 69). A reprint from the general report (preliminary publication) of the third Congress of the International Assn. for Bridge and Structural Engineering (Liège, September 13-18, 1948) discusses the analysis of safety and effect of dynamic forces.

All these papers are easily readable. They are particularly recommended to students and junior engineers for their high educational value.*

Procedure in Determining Footing Areas (LR 46-54)

CHU-KIA WANG, Assistant Professor of Civil Engineering, University of Colorado, gives a method for proportioning footing areas to obtain even settlement.

Nomenclature

A = Area of footing
D = Dead load on the footing
L = Live load on the footing
cr = Subscript for critical footing with the largest value of L/D
k = Portion of live load to be added to dead load to cause even settlement
P_e = Even soil pressure for even settlement
P_t = Allowable total soil bearing pressure
A_{1x} = Area of footing x as required by P_t
A_{2x} = Area of footing x as required by P_e

In the design of footings for buildings, it is necessary to proportion the bearing areas of footings so that as even settlement as possible can be attained. It is generally assumed that there would be even settlement if the bearing pressure under all footings were made nearly even, although many factors other than bearing pressure could more or less affect the amount of settlement. With this proposition accepted, the question arises as to under what loading condition the soil pressure under all footings is to be made even. It is not difficult to see that this loading should not include the full maximum live load used in the design of the superstructure because the settlement is largely caused by loading of long and sustained duration, which may be the sum of dead load and a

*Reprints not available through the American Concrete Institute. Address inquiries to the University of Liège, Liège, Belgium.

portion of the live load depending on the nature of the building. For instance, a larger portion of the live load should be used in a warehouse design, while a smaller portion can be used in a school building design.

The bearing area required at any footing x can be computed in two ways:

$$A_{1x} = \frac{D_x + L_x}{P_t} \qquad \text{or} \qquad A_{2x} = \frac{D_x + kL_x}{P_e}$$

The larger area required should be provided. The procedure of determining P_e is as follows. First compute the ratio of L to D for every footing. The footing with the largest ratio of L to D will be called the "critical footing." Determine A_{cr} required at this footing by

$$A_{cr} = \frac{D_{cr} + L_{cr}}{P_t} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

P_e is then found from:

$$P_e = \frac{D_{cr} + kL_{cr}}{A_{cr}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

It can be proved that A_{2x} is always greater than A_{1x}. Thus

$$A_x = A_{2x} = \frac{D_x + kL_x}{P_e} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The following discussion furnishes this proof.

Given:

$$\frac{L_{cr}}{D_{cr}} > \frac{L_x}{D_x} \qquad A_{cr} = \frac{D_{cr} + L_{cr}}{P_t} \qquad P_e = \frac{D_{cr} + kL_{cr}}{A_{cr}}$$

$$A_{1x} = \frac{D_x + L_x}{P_t} \qquad A_{2x} = \frac{D_x + kL_x}{P_e}$$

To Prove:

$$A_{2x} > A_{1x}$$

Proof:

	Statement	Reason
(1)	$\frac{L_{cr}}{D_{cr}} > \frac{L_x}{D_x}$	Given
(2)	$\frac{D_{cr}}{L_{cr}} < \frac{D_x}{L_x}$	When both fractions are reversed, the sense of the inequality is altered.
(3)	$\frac{D_{cr} + L_{cr}}{L_{cr}} < \frac{D_x + L_x}{L_x}$	Adding 1 to each side of (2)
(4)	$\frac{D_{cr} + L_{cr}}{(1-k)\,L_{cr}} < \frac{D_x + L_x}{(1-k)\,L_x}$	Multiplying each side of (3) by $1/(1-k)$

(5) $\dfrac{(1-k)\,L_{cr}}{D_{cr}+L_{cr}} > \dfrac{(1-k)\,L_x}{D_x+L_x}$ Same reason as (2)

(6) $\dfrac{-D_{cr}-kL_{cr}}{D_{cr}+L_{cr}} > \dfrac{-D_x-kL_x}{D_x+L_x}$ Subtracting 1 from each side of (5)

(7) $\dfrac{D_{cr}+kL_{cr}}{D_{cr}+L_{cr}} < \dfrac{D_x+kL_x}{D_x+L_x}$ When sign of each side is reversed, the sense of the inequality is altered.

(8) $\dfrac{(D_{cr}+kL_{cr})/P_e}{(D_{cr}+L_{cr})/P_t} < \dfrac{(D_x+kL_x)/P_e}{(D_x+L_x)/P_t}$ Multiplying each side of (7) by P_t/P_e

(9) $\dfrac{A_{cr}}{A_{cr}} < \dfrac{A_{2x}}{A_{1x}}$ or $A_{2x} > A_{1x}$ Substituting the given into (8)

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

New river wall, South Bank, Lambeth

Concrete and Constructional Engineering (London)
V. 44, No. 11, Nov. 1949, pp. 349-352

Reviewed by GLENN MURPHY

The construction of a 1700-ft wall from the County Hall to below Waterloo Bridge on the south bank of the Thames is described. The wall, which is constructed of plain concrete faced with Cornish granite, has a base width of 20 ft and a total height of about 45 ft. Nine expansion joints are provided. Construction details are described.

The "Zofra" prestressed concrete railroad tie (El durmiente tipo "Zofra")

ZORISLAV FRANJETIC and M. ROS, *Hormigon Elastico* (Buenos Aires),
V. II, No. 2, July 1949, pp. 10-18

Reviewed by FRANCISCO J. CORDOVA

This is the second and final article describing the "Zofra" type prestressed concrete railroad tie. It contains the mathematical analysis and the statics calculations involved in the design of the structural member. Various analysis diagrams and cross sections of the tie are shown.

Foundations (La tecnica delle fondazioni)

LUIGI SANTARELLA, Ulrico Hoepli, Milan, 1944

Reviewed by GENNARO MIANULLI

The third edition of this textbook has been brought up to date by Dr. Maria Mattiazzo-Santarella. New material has been added and chapters amplified. Spread footings and continuous foundations on solid and filled-in soil are treated very thoroughly. It is unfortunate that the importance of machinery foundations was limited to a few pages and to theoretical considerations only.

Prestressed concrete beams in a building in Edinburgh

Concrete and Constructional Engineering (London),
V. 44, No. 11, Nov. 1949, pp. 353-356

Reviewed by GLENN MURPHY

A 3-story store and distribution building being constructed by the Ministry of Works for H. M. Stationery Office in Edinburgh is of concrete construction. The floors, designed for a live load of 300 psf, are supported by prestressed beams, utilizing the Magnel system of cable reinforcement. The beams were cast on the site, hydraulic jacks being used to apply the prestressing force of 3.35 tons per pair of wire.

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 7, Mar. 1950, *Proceedings*, V. 46. Address 18263 W. McNichols Rd., Detroit 19, Mich. Copies of articles or books reviewed are **not** available through ACI. In most cases they can be obtained direct from the original publishers. Address, when available, will be furnished by ACI on request.

Hydro-electric works in Uganda

Concrete and Constructional Engineering (London), V. 44, No. 11, Nov. 1949, pp. 346-347

Reviewed by GLENN MURPHY

This one-page article accompanied by a page of drawings gives a concise description of the plans for a 2725-ft length dam and appurtenances to be built about 1½ miles below where the river Nile leaves Lake Victoria. The ultimate power development of 150,000 kw will operate under a nominal head of about 72 ft. The estimated quantity of concrete for the entire project, including a 570-ft bridge is 250,000 cu yd. The use of boulders up to 5 tons in weight is contemplated for the concrete.

The design of reinforced concrete structures (La progettazione delle costruzioni in cemento armato)

GIOVANNI ARIOSIO, Ulrico Hoepli, Milan, 1947

Reviewed by GENNARO MIANULLI

This book is intended to further student training in design of reinforced concrete structures. The author solves an unusually large number of problems in foundations, slabs, sheds, rigid frames and water tanks. Practical consideration is given to each problem, the method of solution and difficulties encountered is explained clearly, correlated with the sequence of computation.

This text book is also of value to young graduate engineers.

Plant for prefabrication of structural members (Industria de elementos prefabricados)

Cemento Portland (Buenos Aires), V. IV, No. 20, Aug. 1949, pp. 7-9

Reviewed by FRANCISCO J. CORDOVA

This paper describes a plant recently built by Compania de Edificacion Moderna in the town of Tapiales, Province of Buenos Aires, The plant manufactures structural members for the construction of walls and roof truss. The plant has also facilities for the prefabrication of members for floor structures, beams, electric line posts, etc. Descriptions of the different prefabricated types of roof truss and wall structures are given. Photographs and drawings showing prefabrication methods and structural members are included.

Theory and practice of prestressed concrete (Teoria y practica de la precompresion)

JULIO PIZZETTI, *Hormigon Elastico* (Buenos Aires), V. II, No. 2, July 1949, pp. 19-28

Reviewed by FRANCISCO J. CORDOVA

The author presents a detailed description of his experiences with prestressed concrete. Pizzetti is a follower of the Italian school of prestressing concrete, having worked for many years in Turin with Colonnetti, the Italian authority.

The various methods of prestressing concrete are explained. The physical and structural properties of concrete and steel are described. Two main dangers in prestressing concrete are analyzed, namely, creep and displacement of wire reinforcement due to loss of bond.

Several stress diagrams are shown.

Calculation of the modulus of elasticity of some rocks from the Province of Buenos Aires, Argentina (Determinacion del modulo de elasticidad de algunas rocas de la Provincia de Buenos Aires)

AARON HELFGOT and JOSE LOMBARDI, *Publicacion del Laboratorio de Ensayos de Materiales e Investigaciones Tecnologicas* (La Plata, Argentina), Series II, No. 27, 1948, 23 pp.

Reviewed by FRANCISCO J. CORDOVA

This paper describes laboratory testing of four types of structural stones from the Province of Buenos Aires. The determination of the modulus of elasticity was based on six prisms from each individual rock type which were prepared and tested in accord-

ance with ASTM. The testing procedure is fully described. Diagrams and tables showing results of the tests are shown. An appendix analyzing errors due to observations and calculations is included.

Failures in concrete roads

Concrete and Constructional Engineering (London), V. 44, No. 11, Nov. 1949, pp. 360-362 — Reviewed by GLENN MURPHY

An abstract is given of a section of the report of the Road Research Board for 1946-47. In one case a number of slabs failed after 20 years of satisfactory service as a result of improper maintenance which allowed water to penetrate and soften the subgrade, and overloading during the war. Results of recent tests on these experimental concrete slabs are described. These include slabs from 3 to 8 in. thick and from 13 to 340 ft long. The deleterious effect of wet subgrade is shown. The influence of frost damage is noted. The physics of subgrade moisture is also discussed briefly.

A sewerage scheme in Lancashire

Concrete and Constructional Engineering (London), V. 44, No. 11, Nov. 1949, pp. 357-359 — Reviewed by GLENN MURPHY

The project includes the construction of about 5 miles of sewer line passing under the Leeds and Liverpool canal, under the railway, and alongside the Gladstone graving dock to discharge into the river Mersey. One portion of the line consists of 2 ft and 3 ft concrete drain pipe, while a U-shaped concrete section lined with brick is utilized for about one-third of the length. For the tunnel sections precast reinforced concrete segments lined with brick were used. Conditions varying from hard rock to water logged soil were encountered. In one section precast reinforced concrete piles were used to support the sewer. A tide-flap chamber is provided at the discharge end in the river.

Contractor meets close design tolerances in building

J. J. POLIVKA, *Civil Engineering*, V. 19, No. 1, Jan. 1949, pp. 40-44 — AUTHOR'S SUMMARY

A description of the Podolsko Bridge over the Vltava River in Czechoslovakia, main arch of 498-ft theoretical span, the longest in Central Europe. Long spandrel arches are supported at the quarter points of the main span. Theoretical and experimental analyses have shown that a solid concrete arch of constant thickness (6 ft 7 in. equal to 1.75 of the span) fully satisfies all the requirements of present engineering knowledge. Especially important is the fact that the stresses due to elastic and plastic deformation in such an arch are lower than in an arch with increasing cross section toward the springing points or with a box-shaped section. Analysis of cost ($9.75 per sq ft) is given. Discussion of this article may be found in *Civil Engineering*, Mar., Apr., May, June and Aug., 1949.

Keeping streets free of winter hazards

The Foundation, The Engineering Society of Detroit, V. XIV, No. 5, Jan. 1940, pp. 3-6

The results of a study by a committee of the Engineering Society of Detroit on the use of salt for removing snow and ice from streets and its effects on autos and pavements are reported.

The common methods of treating ice by the application of abrasives, abrasives plus chemicals, calcium chloride, processed sodium chloride or rock salt are considered with the conclusion that rock salt is the most effective and the least expensive to the over-all economy.

To control injury to concrete pavement caused by the use of raw chemicals air-entraining concrete is suggested for pavements and other structures exposed to such conditions.

A butterfly-wing bridge for San Francisco

E. KENDALL THOMPSON, *Architectural Record*, July 1949 — Reviewed by J. J. POLIVKA

A second San Francisco Bay crossing entirely in reinforced concrete proposed by Frank Lloyd Wright and J. J. Polivka was submitted to California's Legislature Committee for further study. A typical standardized unit of the bridge is 160 ft long, center-to-center, with a six-lane roadbed on a central arched girder built on and into the tap-root hollow concrete piers. Thin shells, underneath the roadbed, extend from the lower point of the central girder to the parapets. Tap-root piers are precast, floated to the site and sunk into the natural or filled sand layer which is then solidified from the interior of the piers by injection of chemicals. The passage of shipping without disturbing the flow of traffic is provided either by double row of turn-table spans or by a 1000-ft central span of twin arches with split roadway and central park. This would be the world's longest single-span concrete arch. The cost of the 10-mile long bridge was estimated at $83.8 million as compared with $155 million for a steel copy of the existing Bay bridge. The estimated cost of the butterfly-wing bridge could be further reduced by use of prestressed concrete.

Motor road from the city of Buenos Aires to "Ministro Pistarini" National Airport (Autopista al Aeropuerto Nacional Ministro Pistarini)

Cemento Portland (Buenos Aires), V. IV, No. 20, Aug. 1949, pp. 2-6 — Reviewed by FRANCISCO J. CORDOVA

This article describes design and construction features of the partially completed motor road connecting the city of Buenos Aires and "Ministro Pistarini" National Airport. The expressway is 20.775 kilometers long, of which 5.071 kilometers are within the city limits and the remaining 15.704 kilometers are in the Province of Buenos Aires.

Intersections with railroads and highways are made at different levels and only four crossings at grade were provided within the city limits. The expressway consists of two 3-lane central paved roads, each 10.50 m wide, separated by a center strip varying from 3 to 6 m. Two lateral paved roads for heavy traffic and paths for pedestrian and cyclists are included in the design. Esthetic consideration has been given, not only to the appurtenant structures, but also to the route design and landscape. The road was designed for speeds up to 120 kilometers per hour and a minimum transit capacity of 12,500 vehicles per hour.

The pavement thickness was calculated according to the formulas of Westergaard with variations introduced by Bradbury and the Public Roads Administration. No provision was made for transversal expansion joints. Concrete mixtures were designed so as to attain a modulus of rupture at 28 days of 50 kg per sq cm. This condition was satisfied by the use of concrete mixtures containing 300 kg of cement per cu m and a water-cement ratio varying from 0.44 to 0.48. Coarse aggregate was acquired from quarries at Olavarria, Tandil and Dean Funes (Cordoba). Fine aggregate was imported from Rio Segundo (Cordoba). The cement was manufactured in Argentina.

Photographs and a cross section of the motor road are shown.

Continuous beam (L'asta solidale)

ATTILIO LINARI, Pellerano-Del Gaudio, Naples, 106 pp. Price L 700 ($1.20) — Reviewed by J. J. POLIVKA

Volume I of this concise treatise has the subtitle "Continuous beam on simple supports" and presents a simple method of analyzing continuous beams and simple frames

composed of any number of straight structural members (beams, columns, posts, struts), having throughout constant or different stiffness, with full or partial restraint at the ends, or with hinged supports. The method consists of determining one redundant value called "principal unknown" (incognita principale) from one resulting equation (equazione risolvente). The other statically indeterminate values (bending moments, direct stress and shear) are derived and obtained from numerous tables in a simple accurate and time saving procedure. The method and the elaborate tables can be used for any type of loading. It can be applied (as demonstrated in three other volumes) to : (a) multiple frames (V. II); (b) open-web girders (Vierendeel type) with symmetrical arrangement with respect to the horizontal axis (V. III); (c) continuous beams and multiple frames with variable moment of inertia, polygonal frames and arches (V. III); (d) open-web girders (Vierendeel type) with parallel chords of different stiffness and other structures having similar characteristics, *e.g.*, multiple-story frames, etc. (V. IV). All these structures can be analyzed on the same principle as continuous beams by introducing factors which express elastic yielding of individual joints or imaginary supports. Advantages of Linari's method can be summarized as follows: (1) direct determination of any redundant value (elastic rotation at the ends of any structural member, bending moment, direct stress and shear in any arbitrarily chosen cross section of a structural member) as a function of the moment at the first left joint (or of the respective elastic rotation), and also as a function of the known values (elastic weights or stiffness of members and fixed-end moments due to loading), under proper consideration of elastic displacement at the joints wherever they occur, (2) direct determination of the node points ($M = 0$) and maximum moment in any member, and also of analytic terms necessary to draw influence lines for any of the internal forces, moment direct stress and shear, at any point of each individual member, without solving the entire system, (3) application to any statically indeterminate structure occuring in engineering practice, (4) the solution is exact and speedy, (5) the great number of redundants is reduced to very few, and to only one in most applications, (6) elimination of intermediate unknown and superfluous parameters and (7) the basic results of the analysis can readily be used for other types of superimposed load and for other conditions of restraint at the supports. The preface is written by Prof. Adriano Galli, a noted authority in structural analysis.

Report No. 100 of the German committee for reinforced concrete

Wilhelm Ernst & Sohn, Berlin, 1949, 18 DM plus postage Reviewed by RUDOLPH FISCHL

Tests with reinforced concrete beams to determine the limit value of reinforcement (Versuche an Stahlbetonbalken zur Bestimmung der Bewehrungsgrenze)

W. Gehler, H. Amos, and E. Friedrich

Results of the tests and the Dresdner calculation method for the plastic range of concrete (Die Ergebnisse der Versuche und das Dresdner Rechenverfahren fur den plastischen Betonbereich)

W. Gehler

Part I explains the notations used in this report. Three are especially defined: (1) the load-bearing value of a reinforced concrete beam, $\delta = \dfrac{fb}{f_e/n}$, (2) the two limits of δ, which are $\lim \delta = T_s$, the load-bearing value at failure at yield stress of the steel, and $\lim \delta = T_c$, the load-bearing value at failure at ultimate compression of the concrete, and (3) the limit value of the reinforcement, where $T_c = T_s$, hence both concrete and steel are completely utilized.

Part II reports the extended Dresdner tests in which 188 reinforced concrete beams with 2.40 m span were tested. Four different grades of steel with yield points from 2425 kg per sq cm to 4460 kg per sq cm, and two qualities of concrete with ultimate strength of 120 per sq kg cm and 160 kg per sq cm were used.

The results of the tests showed that it is possible to establish the limit value of reinforcement which separates the under-reinforced beams from the over-reinforced beams, that a safety factor of approximately 12.5 percent was found between the designed and the actual strength of the beams, and that the limit value of reinforcement increases for all grades of steel with the quality of concrete and decreases with increasing yield point of the steel.

The ratio between the moment where the first cracks appear, and the ultimate moment, increases by decreasing ratio of reinforcement.

Part III deals with the test results and the Dresdner calculation method for the plastic range of concrete. The tests proved that the value $n = 15$, generally used in German calculations, is fully satisfactory for under-reinforced beams. The Dresdner calculations assume for a rectangular beam at ultimate load a rectangular stress distribution for the plastic flow. The position of the neutral axis at ultimate stress is to be found after this calculation method by means of the common calculations with $n = 15$. $0.8f'_c$ is used as average compressive stress in the plastic range. By using the quality ratio $k = \frac{f_s/n}{0.8f'_c}$, the so-called compound value $V = 1/k = \frac{0.8\,f'_c}{f_s/n}$ is introduced. Graphs were plotted to evaluate the test results. The use of these relations lead to the development of a simple practical formula for single reinforced rectangular beams: $\mu_G = 4\,V - 0.5$ (in percent), where μ_G expresses the limit value of reinforcement.

The corresponding formula for a single reinforced T-beam is: $\mu_G = c_z V - 0.5$ (in percent), where c is a coefficient dependent on the shape of the T-beam.

An exception is the double reinforced rectangular beam with $A_s = A'_s$, for which case $\mu_G = 2$ percent was found to be a constant value.

By using the compound value V, the limit reinforcement may easily be determined for any grade of reinforcing steel and any quality of concrete. It will be found uneconomical to use high-grade steel with a low quality concrete.

Prestressed concrete used for boldly designed structures in Europe

E. R. SCHOFIELD, *Civil Engineering*, V. 19, No. 9, Sept. 1949, pp. 22-27 Reviewed by J. R. SHANK

European structural engineers are now using prestressed concrete for bridges, buildings, hangars, dam ties, railroad ties, tanks, water pipes, aqueducts, sewers, poles, piles, and runway paving.

High tensile strength steel wires from ¼ in. to 5/64 in. diameter having ultimate strengths as high as 340,000 psi in the latter case were used under applied unit stresses of from 125,000 to 180,000 psi and concretes as high as 9000 psi ultimate strength at 2000 psi working stress are used. Heavy reliance on bond is imposed on the smaller wire.

The slump of the concrete is zero. The placing of this concrete requires full use of both internal and external vibrators whether metal or wood forms are used. Vibration is continued until a film of moisture appears on the top surface.

Much of this construction is prefabricated in easily handled sizes in extensive and elaborate factories. Curing is done at Stockholm in a water basin with water at 160 F. At other places curing is done by live steam at the same general temperature. In the latter case 8000 psi has been obtained at 18 hours. High early strength cement is generally favored.

Much ingenuity is exercised in the design and construction of bridges. Box girder sections with prestressing steel in the bottom flanges have been used. Where the larger wire is used the wires are in sheathing or prefabricated holes until after concrete placing, curing, hardening and prestressing, when they are grouted in. In some cases the mechanical prestressing device is removed after the grouting has hardened. In

other cases these devices are concreted in for permanence. A solid rubber core serves to make holes for prestressing wires. Tension on the core produces progressive lessening of the cross section and so allows its easy removal. A special creamy grout known as "Colcrete" has been developed for grouting the wires.

Eugene Freyssinet has been reconstructing bridges over the Marne River generally of the two-hinged arch type of precast voussoirs weighing from 2 to 4 tons each for spans of 243 ft with ratio of rise to span of 1:45. No centering was used, the voussoirs were held by overhead cables until the arch was sprung.

This European practice is said to save dead weight, prevents concrete cracks, saves steel, concrete and depth of beam structure, increases resistance to flexure, resists fatigue of reinforcing steel and saves time and money.

Statically indeterminate frameworks

THOMAS F. HICKERSON, University of North Carolina Press, Chapel Hill. $5.00

Reviewed by W. E. HANSON

Previous editions of the book appeared in 1934 and 1937; the original title was "Structural Frameworks." New material has been added in this third edition while certain topics have been omitted, so that the total number of pages remains almost the same as that of the second edition.

The reader will find pronounced emphasis placed on the author's "single cycle" method of determining bending moments in all types of continuous statically indeterminate beams and frames. Some of the illustrative problems include comparative solutions by the Hardy Cross moment distribution method.

Briefly, the author's principal method for the analysis of continuous structures is as follows. The "stiffness" of a member involves not only E, I, and L but also the restraining conditions or fixity of the far end. The "degree of fixation" at the end of any member is determined by the "stiffnesses" of the other members rigidly attached to the joint. The "degree of fixation" always lies between zero for a hinge and unity for complete fixity. The final moment at the end of a member is then found from an expression involving the bending moments at both ends if they are 100 percent fixed and the existing "degree of fixation" of each end. After determining the final moment at the end of a member which frames into a multi-member joint, this moment is then "balanced" into adjacent members in accordance with the "stiffnesses" of these members. The author's "carry-over factor," which determines the amount of moment carried to the far end of an adjacent member which has received its proportionate share of "balanced" moment, is found by an expression which involves the "degree of fixation" of the far end.

In general, if the continuous structure consists of several loaded spans, one loaded span at a time is considered and the moments resulting from loads in that particular span are found by the author's "single-cycle" method. The final moments produced by all the loads acting on the structure are then found by superposition.

The book is divided into two parts and three appendixes. Part I deals with members with constant moment of inertia while Part II covers members whose moments of inertia are variable. The appendixes make up more than half the book. Appendix I presents 76 tables of various coefficients covering a wide range of type of structure and loading conditions. Although many of the tables give coefficients designed to facilitate the application of the author's "single cycle" method, a number of the tables and coefficients can be useful in other methods of anslysis. Appendix II contains two illustrative examples showing the analysis of a tall building frame subjected to wind load. Two approximate methods—the cantilever and portal methods—are discussed and used in the analysis. Appendix III gives the derivation of formulas for beam and load coefficients.

The new cement factory of Skanska Cementaktiebolaget at Stora Vika

LOVHOLMEN D. WERNER, *Betong* (Stockholm), V. 34, No. 3, 1949, pp. 111-148 AUTHOR'S SUMMARY

The descriptive matter is preceded by an account of how the consumption of cement has developed in Sweden from the beginning of the century. The fluctuations are analyzed and placed in relation to different periods of booms and slackness in business and construction during this time.

The production and sale conditions of the Swedish cement industry during the second world war and in the immediate post-war period are reviewed. In this connection are described the difficulties set up by the import embargo, and the extraordinary measures that had to be adopted by the cement industry to keep up production. The scant supply of imported fossil fuel had to be supplemented with domestic fuels, which under normal conditions are less economical. Fuel saving devices were further applied to the operation of the rotary kilns, and a special substitute cement was manufactured containing some unburnt material and produced by a special grinding process.

These measures in combination enabled Skanska Cementaktiebolaget to save about 65 percent of the imported coal that before the war would have been required to produce a corresponding quantity of cement. Some other materials and supplies which could not be obtained were replaced by products not used in normal times.

In spite of the difficulties which characterized this period the country's requirements of cement increased greatly. To satisfy this augmented demand the capacity of the Koping works was doubled, and long before the end of the war plans were prepared for a new cement factory.

After thorough investigation and planning it was decided to locate the new factory at Stora Vika, about 35 miles south of Stockholm, whence the cement was to be shipped in bulk in special vessels to Stockholm where a specially constructed packing and shipping plant was to be erected. These dispositions were conditioned on Stockholm and environs being the country's most concentrated cement consuming region.

A detailed description is given of the new cement factory. The most important raw materials are primary rock limestone and clay. The quarrying, loading and shipment of the limestone is done with large, modern machines. The fine-crushed and graded limestone from up-to-date machines is conveyed by an automatically operated aerial ropeway a scant kilometer to the factory.

The raw material is ground in two stages between which is installed flotation concentration for removing magnesium silicate impurities in the limestone. The burning is done by the wet process in 2 rotary kilns 145 meters long with a total capacity of about 1000 tons of cement clinker per day. The cement is ground in modern internal watercooled mills, and the finished cement is conveyed pneumatically to storage bins at the newly built deep-water harbor which is also equipped for the unloading of coal, gypsum, etc. A large storage house 275 meters long and 35 meters wide, was built at the harbor for limestone, gypsum, correction materials, clinker and coal. All materials are handled by two 10-ton traveling cranes.

Auxiliary houses and plants were also built and a residential area for employees was laid out. The cement is carried to Stockholm in a 1700-ton motorship equipped with newly designed fully automatic loading and discharging devices which reduce manual labor to a scant 3 percent of that required for manual loading and discharging of cement in sacks.

The Stockholm storage and distributing plant has bins for storing a winter's supply of cement for the district. It is equipped with sacking machines and appliances for shipping both bulk and sacked cement direct to the point of consumption. Special tank trucks are used for conveying bulk cement the consumption of which has increased 30 percent in the Stockholm region.